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GEOTECHNICAL DATA BANK FOR INDIANA

Y-K. T. Lo

PURDUE UNIVERSITY
DIANA STATE HIGHWAY COMMISSION

80-7

Final Report

GEOTECHNICAL DATA BANK FOR INDIANA

TO: H. L. Michael, Director
Joint Highway Research Project

May 14, 1980

Project: C-36-51T

FROM: C. W. Lovell, Research Engineer
Joint Highway Research Project

File: 1-5-20

Attached is the Final Report on the HPR Part I Study titled "A Computerized Information Storage and Retrieval System for the Soils of Indiana". The report is entitled "Geotechnical Data Bank for Indiana" and is authored by T-K. T. Lo of our staff.

The research involved the storage of about 7500 data sets and selected statistical interpretations of them. These included regression equations to predict soil properties and formulation of statistical soil profiles for a variety of soil areas in Indiana. Examples of how to use the data bank in routine ISHC operations were also developed and reported.

The Report is submitted for review, comment and acceptance in fulfillment of the objectives of the referenced HPR study.

Respectfully submitted,

C. W. Lovell/HLM

C. W. Lovell
Research Engineer

CWL:ms

cc: A. G. Altschaeffl
W. L. Dolch
R. L. Eskew
G. D. Gibson
W. H. Goetz
M. J. Gutzwiller
G. K. Hallock
D. E. Hancher
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C. F. Scholer
K. C. Sinha
C. A. Venable
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L. E. Wood
E. J. Yoder
S. R. Yoder

Final Report

GEOTECHNICAL DATA BANK FOR INDIANA

by

Y-K. T. Lo
Graduate Instructor in Research

Joint Highway Research Project

Project No.: C-36-51T

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16. Abstract

Nearly all readily accessible geotechnical data have been stored, bringing the data base to nearly 10000 sets. Several types of statistical operations were undertaken on the data, including median model characterizations of frequency distributions and regression analysis to produce soil parameter prediction.

The most difficult aspect of statistical operation on the data is selection of the appropriate data population or grouping. Data have been treated in the following groups: state-wide, physiographic section, parent material area, AASHTO classification, and certain combinations thereof. The proper grouping seems to vary with the frequency distribution being examined or the prediction being attempted.

Selected usage of the data bank is recommended via specific examples. Further research is recommended and is planned by the cooperating agencies.

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NOMENCLATURE

English

Symbol	Represents
AASH(T)O	American Association of State Highway (and Transportation) Officials
ANOV	Analysis of variance
Bedrk	Depth to bedrock
CBR	California bearing ratio
CBR S01	Soaked California bearing ratio at 100% of standard Proctor maximum dry density
CBR S02	Soaked California bearing ratio at 95% of standard Proctor maximum dry density
C_c	Compression index
CDC	Control Data Corporation
CI	Consistency index
C.I.	Confidence interval
C_r	Recompression index
C'_r	Compression ratio
c_u	Undrained shear strength
c_v	Coefficient of consolidation

Symbol	Represents
c	Vane shear strength or cohesion as specified
c'	Effective cohesion
D	Specimen diameter
DIF	Data input form
D_r	Relative dry density
d.f.	Degree of freedom
E	Modulus of elasticity
El	Elevation
e	void ratio
e_o	initial void ratio
FM	Full model
G	Specific gravity
GI	Group index
IR	Interquartile range
Ip	Plasticity index
ISHC	Indiana State Highway Commission
LI	Liquidity index
MSE	Mean square due to error
MSR	Mean square due to regression
m	a factor (= 0.127 E/c)

Symbol	Represents
m.s.	Mean square
N	SPT values (resistance)
N_q	Terzaghi bearing capacity factor
n	Number of observations (samples)
OMC	Optimum moisture content
P	Failure load under plate load test
PCA	Portland Cement Association
PFRA	The Prairie Farm Rehabilitation Administration
PUCC	Purdue University Computing Center
p_c	Preconsolidation pressure
p_o	Overburden pressure
p'	Mean principal effective stress $(= 1/3\ (\sigma_1' + 2\sigma_3'))$
q	Deviator stress $(= \sigma_1 - \sigma_3)$
q_u	Unconfined compressive strength
$\hat{q}_c$	Estimated compressive strength
R	Correlation coefficient
$\lvert R \rvert$	Multiple correlation coefficient
R^2	Square of (multiple) correlation coefficient
RM	Reduced model

Symbol	Represents
S	Degree of saturation
Si	Initial degree of saturation
SL	Shrinkage limit
SPSS	Statistical Package for the Social Sciences
SPT	Standard penetration test
s	Suction (potential capillary rise)
sand	Percentage of sand by weight
silt	Percentage of silt by weight
s.d.(S.D.)	Standard deviation
s.d. of est.	Standard deviation of estimate
s.s.	Sum of square
Type	Type of strength tests
$t_{.25}$	25th percentile
$t_{.50}$	sample median
$t_{.75}$	75th percentile
UNIF	Unified soil classification system
U.S.D.A.	United States Department of Agriculture
w	Water content or moisture content
w_L	Liquid limit
w_n	Natural moisture content
w_p	Plastic limit

Symbol	Represents
Greek	
Θ θ theta	
θ	Population median
$\hat{\theta}$	Estimated population median (sample median)
Mμ mu	
μ	population mean
$\hat{\mu}$	sample mean
P ρ rho	
ρ_d	Natural dry density
ρ_{dmax}	Standard Proctor maximum dry density
Σ σ sigma	
σ_1', σ_3'	Effective stresses
σ_1'	Vertical effective stress
σ_3'	Cell pressure less background pore pressure in fully drained triaxial test
σ_{ff}	Normal stress on the failure plane at failure
T τ tau	
τ_{ff}	Shear stress on the failure plane at failure

Symbol	Represents
Φ ϕ phi	
ϕ	Strength angle
ϕ'	effective stress strength angle

HIGHLIGHT SUMMARY

Efforts to extract the geotechnical information contained in the subsurface investigation reports held in the files of the Indiana State Highway Commission (ISHC) have extended over several years. A total of 9442 sets of data have been stored through early 1978 to mid 1979.

Both conventional and nonparametric statistical methods were employed. The data were grouped by using physiographic region or engineering soil classification, or soil association, or a combination of them. One-way classification, two-way classification, and factorial experiment layouts were used to examine the distributions of the data. Regression analysis was used to investigate the functional relationships between design parameters and index properties.

Topographic features were found to be significantly different among physiographic regions. Remolded soil characteristics were also evaluated and contrasted between

physiographic regions but also within AASHTO classifications. Statistical soil profiles were developed showing the relations of soil characteristics versus depth for soil association within a physiographic region. Finally, soil design parameters were estimated by using index properties.

CHAPTER 1 INTRODUCTION

The need for knowledge of pedological and engineering soils information for use in planning, site selection, design, construction and maintenance of transportation facilities is recognized by civil engineers. Data are necessarily limited in quantity and quality due to economic and time constraints. While large amounts of detailed soil data are often available from work performed on adjacent or nearby projects, these data are usually not readily accessible for use or their existence is unknown.

The accumulation of laboratory and field test data by the Indiana State Highway Commission for characterizing the engineering properties of Indiana soils is extensive. The information is retained in the form of subsurface investigation reports, prepared by or for consulting design firms and governmental agencies for use in routine soil investigations. In its present voluminous form, the majority of these data are not very useful. The need exists to make this information more accessible for the engineer interested in detailed information of a site and the engineer interested in soil characteristics over a larger region.

In July 1977, a proposal was submitted to develop and test a computerized information storage and retrieval system for soils in Indiana. The specific objectives of the research effort were to:

1. Identify the sources of individual soil sample data and devise a system for soils data collection and codification.
2. Develop computer information storage programs which are flexible enough to meet the data needs of both the Joint Highway Research Project and the Indiana State Highway Commission.
3. Collect and store data for approximately 7500 soil samples from existing subsurface investigation reports.
4. Statistically correlate significant soil properties with standard soil types and develop interrelationships between selected soil properties.

To achieve these objectives the project was divided into two phases. In the first phase, the input format and computer storage and retrieval systems were developed, and 2508 soil data sets were placed in the bank. Gary Goldberg prepared an Interim Report, JHRP-78-6, dated June 1, 1978, entitled "The Development of the Computerized Geotechnical Data Bank for the State of Indiana" (35). As a part of this phase, a complete instructional user's manual was prepared. Limited statistical analysis of

stored data to January 1978 indicated that grouping of soils by physiographic regions and parent material areas appeared to be justified. The twelve parent material designations used to correlate the engineering soil test data were identified relative to the pedological soil series on an appropriate map. The engineering soils-parent material map for Indiana (62) was not used because of its very small scale and an inability to establish the locations of specific boring sites on it.

Phase II of the research is the subject of this report. It involved the storage of 6934 additional data sets, for a total of 9442 data sets as of December 1979. These data sets were from roadway soil boring reports and from those boring reports for bridge and culvert sites that contained laboratory test data. The bridge and culvert site boring reports were included as an extension and expansion of the project scope in March 1979. It was intended as a part of this extension that standard penetration test data at bridge and culvert sites would also be stored. The gathering of the data for the bridge and culvert sites extended until June 1979. At this time it was discontinued, since the costs of adding the standard penetration data were deemed excessive.

The purposes of Phase II of the study were:

1. to complete the storage of readily available engineering soil test data;

2. to show how to manage the data bank;
3. to evaluate the information stored, and develop correlations and quantitative values for planning and preliminary design by using statistical methods.

Both conventional and nonparametric statistical methods were employed, as discussed in Section 3.3. However, the nonparametric statistical methods appear to fit and explain the varieties of soil characteristics in a superior way. This is further explained in Section 3.3.1.a.

The data were grouped by using physiographic regions, engineering soil classifications, soil associations, or a combination of them. One-way classification, two-way classification, and factorial experiment layouts were used to examine the distributions of the data. Regression analysis was used to investigate the functional relationships between design parameters and index properties. The results are shown in Appendices A-I through A-IV. The soils data were extremely variable in their characteristics, therefore choices of suitable groupings for study were the most difficult tasks in this investigation.

It is emphasized that the samples of soil and topographic characteristics are not uniformly distributed throughout a physiographic region. The soil data distribution map (Figure 3-1) should be consulted before attempting any interpretations of results.

CHAPTER 2 PREVIOUS INVESTIGATIONS

2.1 The Concept of a Geotechnical Data Bank

Each year an enormous amount of geotechnical data generated by field and laboratory measurement are accumulated in the files of public agencies and private consultants throughout the United States and abroad. The data are contained in formal reports of subsurface investigations, and are checked and verified. These data are used in different functions, such as site selection, planning, design, construction and maintenance of new facilities, and reconstruction. Once the project is completed, further use of these data is limited because of their format.

In the State of South Dakota it was found (22) that many expensive soil testing programs were being conducted on new highway routes which were either close to or parallel to the old alignments on which extensive soil testing had been previously undertaken. In 1965 the South Dakota Department of Highways, in cooperation with the Soil Conservation Service, began a program to collect the accumulated geotechnical data from previous projects (22). The data were then stored in a computerized system

and analyzed by using statistical methods for the purposes of planning, location and preliminary design.

Spradling (109) developed a computerized data storage and retrieval system for the State of Kentucky. The Kentucky Department of Transportation devised an extensive coding system for data which were collected but not suitable for direct computer storage. Due to the completeness of this coding system, and its applicability to soil information in general, some of its details were adopted for the Indiana data bank (35).

Recently, the U.S. Department of Transportation published a state-of-the-art report which documented basic information on automatic data processing techniques used by eight states (Colorado, Georgia, Illinois, Louisiana, Minnesota, New York, Pennsylvania, and West Virginia) in managing test data for highway materials (24). Three basic data processing techniques, viz., batch information systems, on-line interactive information systems, and on-line interactive laboratory information systems, were in use.

The Prairie Farm Rehabilitation Administration (PFRA) of Canada collected data on the geotechnical properties of glacial till deposits, glacial lake deposits, and alluvial deposits from many of its previous projects in western Canada (82, 88). A number of empirical relationships between routine classification tests and consolidation

and strength characteristics were developed, as discussed in Section 2.2. The correlations were generated primarily to aid in evaluating information from new sites or to approximate geotechnical information for preliminary investigations.

Similar geotechnical data storage and retrieval systems were developed in the following countries: Sweden (58), Finland and Denmark (58), France (53), Rhodesia (41), Algeria (87), and South Africa (116).

2.2 Empirical Relationships in the Practice of Geotechnical Engineering

2.2.1. Consolidation Parameters:

Consolidation is defined as the process of densification of a soil under sustained loading caused by the expulsion of water from the pores. The laboratory oedometer test is used to determine the consolidation parameters, such as compression index (C_c), recompression index (C_r), and preconsolidation pressure (p_c). Schmertmann (96) and others (52, 81) have recommended that initial void ratio (e_o), compression index (C_c), recompression index (C_r), overburden pressure (p_o), and preconsolidation pressure (p_c) be used to construct the field virgin compression curves and to predict settlements of clay soils in

situ. For details of these constructions refer to (52, 81, 96).

There were also many attempts to predict the consolidation parameters from easy-to-measure soil indices, such as natural moisture content, liquid limit, and initial void ratio. A survey of these attempts is presented in the following sections.

2.2.1. a Compression Index (C_c). The compression index (C_c) has often been correlated with either the liquid limit (w_L), the natural moisture content (w_n), initial void ratio (e_o), or a combination of these. Table 2-1 shows a summary of available regression equations, together with their region of applicability, for the prediction of C_c.

Wroth and Wood (123) proposed from experience that

$$w + A \ln c_u = \text{constant}$$

where w is moisture content, A is a constant, and c_u is the undrained shear strength. Also, from phase relations, $V = 1 + e = 1 + Gw$ for saturated soils, where V is the specific volume, i.e., volume of total mass/volume of solids, e is the void ratio, G is the specific gravity,

Table 2-1. Summary of Published Regression Equations for Prediction of Compression Index, C_c

Regression Equation	Region of Applicability	Reference
$C_c = 0.007\ (w_L - 10)$	Remolded clays	Skempton, (103)
$C_c = 17.66 \times 10^{-5} w_n^{\,2} + 5.93 \times 10^{-3} w_n - 1.35 \times 10^{-1}$	Chicago clays	Peck et al (79)
$C_c = 1.15\ (e_o - 0.35)$	All clays	Nishida (76)
$C_c = 0.30\ (e_o - 0.27)$	Inorganic, cohesive soil; silt, some clays; silty clays; clay	Hough (43)
$C_c = 1.15 \times 10^{-2} w_n$	Organic soils	Moran, Proctor, Mueser & Rutledge, (68)
$C_c = 0.256 + 0.00106(w_L - 65) + 0.32(e_o - 0.84) \pm 0.063$	Heavy medium clays and silts in Brazil	Cozzolino (21)

Table 2-1. (continued)

Regression Equation	Region of Applicability	Reference
$C_c = 1.21 + 0.0072(w_L - 95) + 0.53(e_o - 1.87) \pm 0.32$	Soft clays and silts in Brazil	Cozzolino (21)
$C_c = 0.545\ (e_o - 0.756)$	Peats in Japan	Watanabi et al (119)
$C_c = 0.016\ (w_L - 60)$	Tokyo subsoils	Ohsaki (77)
$C_c = 0.086\ w_n$ $C_c = 0.5(e_o - 1)$	Peats in Japan	Maegnchi et al (60)
$C_c = 0.009\ (w_L - 10)$	Normally consolidated clay	Terzaghi et al (113)
$C_c = 0.21\ e_o + 0.0111\ w_p - 0.17$ $C_c = 0.21\ e_o + 0.00314\ w_L - 0.07$	Indian soils	Sengupta et al (98)

Table 2-1. (continued)

Regression Equation	Region of Applicability	Reference
$C_c = (0.0075 \sim 0.011)\ w_n$	Canadian soils	NRCC (73)
$C_c = (0.45 \sim 0.75)\ e_o$		
$C_c = 0.75\ (e_o - 0.5)$	Soils of very low plasticity	Sowers (108)
$C_c = 0.01\ w_n$	Chicago clays	Osterberg (78)
$C_c = 0.0087\ w_L - 0.089$	Tamil Nadn clays in India	Raghavan et al (85)
$C_c = 0.39\ e_o - 0.03$		
$C_c = 0.0093\ w_n + 0.0142 \simeq 0.9\ \frac{w_n}{100}$	Tamil Nadn clays in India	Raghavan et al (85)
$C_c = 0.0047\ w_L + 0.3473\ e_o - 0.2312$		
$C_c = 0.0033\ w_L + 0.0086\ w_n - 0.1323$		

Table 2-1. (continued)

Regression Equation	Region of Applicability	Reference
$C_c + 0.40\ (e_o - 0.25)$	Greek and U.S. soils	Azzouz et al (6)
$C_c = 0.01\ (w_n - 5)$		
$C_c = 0.006\ (w_L - 9)$		
$C_c = 0.37\ (e_o + 0.003\ w_L - 0.34)$		
$C_c = 0.40\ (e_o + 0.001\ w_n - 0.25)$		
$C_c = 0.009\ w_n + 0.002\ w_L - 0.10$		
$C_c = 0.37\ (e_o + 0.003\ w_L + 0.0004\ w_n - 0.34)$		
$C_c = 0.013\ (w_n - 7)$	Peaty soils in Hokkaido, Japan	Kogure et al (49)
$C_c = 0.00782\ w_n^{1.07}$		
$C_c = 0.62\ (e_o - 0.56)$		
$C_c = 0.370\ e_o^{1.17}$		
$C_c = 0.621\ \rho_d^{-1.37}$ (ρ_d: natural dry density in kg/cm^3)		

and w is the moisture content expressed as a ratio for saturated soils. Adding the critical state relationships of Roscoe et al (91), and the assumption that the undrained shear strength at the plastic limit is 100 times that at the liquid limit, Skempton and Northey (104), Wroth and Wood went through a series of rational procedures and derived the following relationship between compression index (C_c), plasticity index (I_p), and specific gravity (G) for remolded soils

$$C_c = \frac{1}{2} I_p G$$

For G = 2.7,

$$C_c = 1.35 I_p$$

The design charts developed by the Canadian Prairie Farm Rehabilitation Administration (PFRA) also demonstrated the relationships of the compression index (C_c) versus natural moisture content (w_n), liquid limit (w_L), and natural dry density (ρ_d) for the soils in western Canada (82, 88).

2.2.1. b <u>Compression Ratio (C_r')</u>. The compression ratio (C_r') is defined as $C_c/(1 + e_o)$. By examining this expression and Table 2.1 it can be concluded that: (1)

there is a correlation between compression index (C_c) and compression ratio (C_r') and; (2) compression ratio is correlated with liquid limit, moisture content, initial void ratio, or a combination of these.

Rutledge (93) and Fadum (29) showed that as the natural moisture content increases, the compression ratio increases linearly for normally consolidated clays. Table 2-2 gives a summary of available regression equations, together with their geographic region of applicability, for the prediction of C_r'.

2.2.1. c <u>Preconsolidation Pressure (p_c)</u>. By definition, the preconsolidation pressure (p_c) is the greatest effective pressure the soil has carried in the past. Preconsolidation may be caused by a variety of factors (52) including:

(1) Removal of overburden

(2) Fluctuations in the groundwater table

(3) Cold-welding of mineral points between particles

(4) Exchange of cations

(5) Precipitation of cementing agents

(6) Geochemical processes caused by weathering

(7) Delayed compression

(8) Tectonic forces due to movements in the earth's crust.

Table 2-2. Summary of Published Regression Equations for Prediction of Compression Ratio, C'_r

Regression Equation	Region of Applicability	Reference
$C'_r = 0.208\ e_o + 0.0083$	Chicago clays	Peck et al (79)
$C'_r = 0.156\ e_o + 0.0107$	All clays	Elnaggar et al (27)
$C'_r = 0.14\ (e_o + 0.007)$	Greek and U.S. soils	Azzouz et al (6)
$C'_r = 0.003\ (w_n + 7)$		
$C'_r = 0.002\ (w_L + 9)$		
$C'_r = 0.126\ (e_o + 0.003\ w_L - 0.06)$		
$C'_r = 0.142\ (e_o - 0.0009\ w_n + 0.006)$		
$C'_r = 0.003\ w_n + 0.0006\ w_L + 0.004$		
$C'_r = 0.135\ (e_o + 0.01\ w_L - 0.002\ w_n - 0.06)$		

Kogure and Ohira (49) proposed the following relationships:

$$p_c = 43.9\ w_n^{-0.913}\ ;(R = \text{correlation coefficient} = 0.821)$$

and

$$p_c = 1.65\ e_o^{-0.988}\ ;(R = \text{correlation coefficeint} = -0.810)$$

where w_n is the natural moisture content, and e_o the initial void ratio for peat soil and underlying clays of the Ishikari area of Hokkaido, Japan.

The Canadian PFRA (82,88) described a relationship between liquidity index (Ll) and Log p_c, where as Ll increases, Log p_c decreases linearly for the soils in western Canada. An examination of this relationship shows that the data pattern appears scattered; the standard error of estimate is accordingly large. The U.S. Navy (118) proposed a similar relationship between LI and Log p_c, with consideration of the dependence of preconsolidation pressure on the soil sensitivity. Bjerrum (11) developed a relationship between the (preconsolidation pressure/overburden pressure) ratio and plastic index for late glacial and post glacial clays.

2.2.2. Compaction Parameters

Compaction is defined as the densification of a soil by means of mechanical manipulation at constant water content, and is measured quantitatively in terms of dry density of the soil.

Proctor (84) demonstrated that there is a definite relationship between density and moisture content. The characteristic peak in the Proctor curve is known as the maximum dry density and its corresponding moisture content as optimum moisture content (OMC). Compaction specifications are written in terms of percentage compaction, a common requirement being 95% to 100%. Percentage compaction is based on the ratio of the field density to the maximum dry density obtained with a specified compaction effort, such as the standard AASHTO or modified AASHTO test, in the laboratory. Thus, the maximum dry density is often regarded as the total objective of the compaction operation.

2.2.2. a <u>Optimum Moisture Content (OMC) and Maximum Dry Density ($\rho_{d\ max}$)</u>. The plasticity characteristics of a cohesive soil reflect its ability to hold water. The less tightly absorbed or highly oriented the water, the greater the freedom of movement for solids in the compaction process (50). Therefore, as the liquid limit, or plastic limit, or plastic index increases, OMC increases and

$\rho_{d\ max}$ decreases. Furthermore, as the optimum moisture content increases, the maximum dry density decreases. These relationships were investigated and verified by Woods and Litehiser (122), the U.S. Navy (118), Narayama Murty (69), and PFRA (82, 88) for a variety of soils. Table 2-3 shows a summary of regression equations, together with their geographical regions of applicability, for the prediction of OMC and $\rho_{d\ max}$.

2.2.2. b <u>California Bearing Ratio</u> (CBR). The CBR test is used to provide a low deformation measure of strength of compacted subgrade soil and is used with empirical curves to design asphalt pavement structures. In the literature, CBR values have been predicted by means of index properties, strength characteristics, and soil classification units.

(1) <u>Evaluation of CBR values in terms of index properties.</u> A relationship between the CBR and the group index (GI) was suggested by the Asphalt Institute (4), and later by Gawith and Perrin (31). Both CBR values were measured at 90% modified AASHTO maximum dry density. As the group index increases, the CBR decreases. Gawith and Perrin (31) also suggested the following equations for the prediction of the CBR values for Australian soils:

$$\text{Log CBR} = 1.886 - 0.0143D - 0.00045A + 0.00515\,\frac{B}{A} - 0.0000456\left(\frac{B}{A}\right)^2 - 0.0037E$$

Table 2-3. Summary of Regression Equations for Prediction of Optimum Moisture Content (OMC) and Maximum Dry Density ($\rho_{d\ max}$)

Regression Equation	Region of Applicability	Reference
$\frac{K}{Dw} - \frac{1}{G} = \frac{x}{100}\left(-\frac{k}{Dw}\right) + \frac{1}{10\sqrt{m} + m(x - x_o)}$ where K is density of water, Dw wet density of soil, G specific gravity, m constant, x_o moisture percentage, x optimum moisture content, and m obtained from $\frac{K}{Dm} - \frac{1}{G} = \frac{x_o}{100} + \frac{1}{10\sqrt{m}}$, Dm maximum dry density.	all soils	Dewan (25)
$OMC = \frac{w_L}{2} \pm \sqrt{\frac{w_L^2}{2}\left\{\frac{0.28\ I_p + 0.495\,x}{\sqrt{w_L\ (w_L - I_p)}}\right\}^2}$	Indian soils	Narayana Murty (71)

Table 2-3. (continued)

Regression Equation	Region of Applicability	Reference
$OMC = 1.0\ w_p - 4.0$	Israel soils	Kassiff et al (48)
$OMC = 1/3\ (w_L + 15)$	Indian soils	Ramiah et al (86)
$\rho_{d\ max} = 2100 - 7.0\ w_L$ (kg/cu. meter)	Israel soils	Kassiff et al (48)
$\rho_{d\ max} = 2125 - 10\ w_L$ (kg/cu. meter)	Indian soils	Ramiah et al (86)

where A is the percentage passing the No. 36 B.S. (British Standard) sieve, B is the percentage passing the No. 200 B.S. sieve, D the plasticity index and E the percentage passing the No. 7 B.S. sieve.

and

$$CBR = 4.5 + \frac{(20 - GI)^2}{18}$$

where GI is the group index.

Stephenson, Karrah and Koplon (110) proposed the following equation to evaluate the CBR values for Alabama soils:

$$\text{Log CBR} = 2.334984 - 0.002425x_1 - 0.006920x_2$$

where $x_1 = \%4 + \%10 + \%40 + \%\ 60 + \%200$

(% of total sample passing each sieve size as indicated),

and $x_2 = \%$ clay

(% of No. 10 fraction as determined by elutriation test).

Kassiff, et al (48) found that the CBR values increase with decrease in the difference between w_p and SL, and with increase in surcharge for Israel soils.

(2) Evaluation of CBR values in terms of strength characteristics. Robinson and Lewis (90) showed that a relationship existed between the CBR value and failure load P (in lb.) of a 3-inch square plate pushed into the ground. They proposed that:

$$\text{Log CBR} = 0.76837 \log P - 1.6442$$

Wiseman and Zeitlen (121) showed that:

$$CBR = mc$$

where c is the vane shear strength,

m is a factor equal to 0.127 E/c

and E the modulus of elasticity of the soil.

The expression was taken from elastic theory, where the settlement of the CBR piston was calculated using a Poisson's ratio of 0.5. With the data from in-situ tests under existing pavements in Israel they suggested the following relationship:

$$CBR = 4.2c - 0.2$$

where CBR is expressed in percent, and the strength, c, in kg/cm^2, is measured from the vane test.

A rational approach, known as the suction method was proposed by Black (12), to estimate CBR values from plasticity and consistency indices. Consistency index (CI) = $(w_L - w_n)/I_p$ where w_L is the liquid limit, w_n moisture content, and I_p plasticity index. This estimation was based on the assumption that at 0.1 inch penetration the soil is close to failure, and that the CBR is equal to sNq/10, where s is suction and Nq the Terzaghi's bearing capacity factor using an effective friction angle. From a knowledge of correlations between plasticity characteristics, suction, and effective angle of friction for saturated British clays, which follow the relationship

$I_p = 0.83\ w_L - 14.2$, and through a process of interpolation and extrapolation Black developed expressions for the CBR. An experimental relationship between unsaturated and saturated CBR values was also developed. Recently Black (13) assumed that CBR = $c_u/23$, where c_u is the undrained shear strength in kPa, and used a similar approach to attain the relationships between c_u, CBR, and plasticity index.

Livneh et al (55) and Greenstein et al (37) used a similar approach to predict CBR values using index characteristics for expansive clays and dune sands.

(3) Evaluation of CBR by Soil Classifications. The four principal variables affecting compacted CBR are soil density, moisture content, compaction effort and the texture and grading of the soil (1). The AASHTO and Unified soil classification systems are based on the textural characteristics and the plasticity characteristics of soil. Accordingly these two engineering soil classification systems have been widely used for the prediction of CBR values.

The U.S. Navy (118) correlated the values of typical characteristics, such as maximum dry density, optimum moisture content, and CBR of compacted materials against the Unified soil classification system. The American Hoist and Derrick Company (1) used the AASHTO classification system, along with qualitative descriptions of soil characteristics, to approximate CBR values.

Casagrande (17) suggested the interrelationship of the Unified system, Public Roads system (which later became AASHO and AASHTO systems), and CBR values. Similar interrelationships were also given by the Asphalt Institute (5) and the Portland Cement Association (83). Having summarized all these interrelationships and made a comparison of groups in the AASHTO and Unified soil classification system, Liu (54) proposed the approximate relative relationships of various groups of both systems to an empirical measure of the CBR values shown in Figure 2-1.

In Israel the AASHTO soil classification affords the best prediction for CBR values. The Canadians have also adopted the AASHTO soil classification, as quoted by Sharma (99), and used group index as well in the final correlation for predicting CBR values.

2.2.3. Strength Parameters

Shear strength is usually assumed to be made up of:

(i) a frictional component which increases linearly with the normal stress on the failure plane, and

(ii) a cohesive component which is constant.

The Mohr-Coulomb equation is ordinarily used to describe the strength and is merely the equation of a straight line:

$$\tau_{ff} = c + \sigma_{ff} \tan \phi$$

where τ_{ff} is shear strength or shear stress on the failure

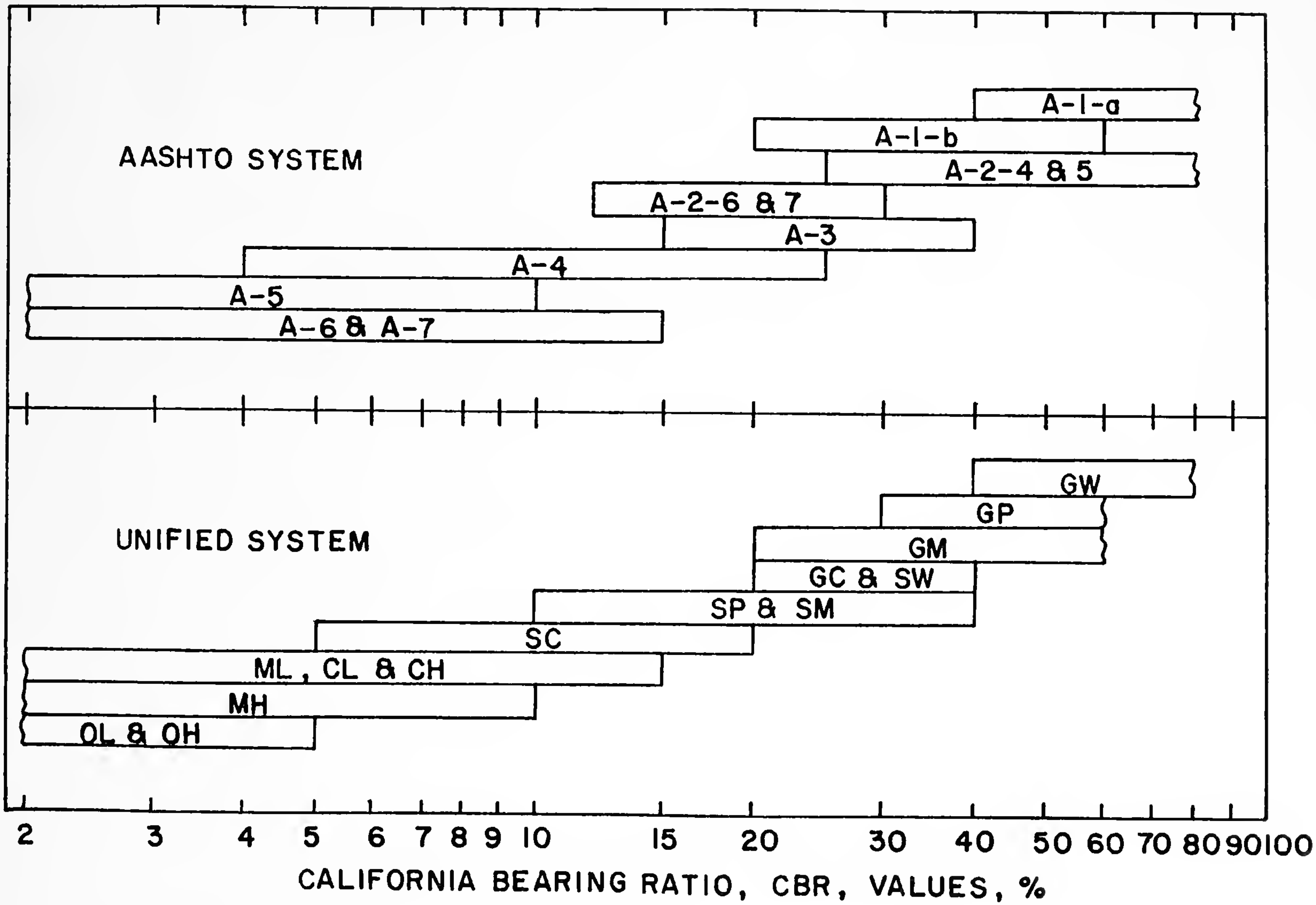

Figure 2-1. Approximate Interrelationships of Soil Classifications and Soaked California Bearing Ratio (after Liu (54))

plane at failure, c the cohesion or strength intercept, ϕ_{ff} normal stress on the failure plane at failure, and ϕ the strength angle. The equation is most rigorously written in terms of effective stress (ϕ', c', σ'):

$$\tau_{ff} = c' + \sigma'_{ff} \tan \phi'.$$

This relationship between shear strength and effective stress, first suggested by Terzaghi, is found to be valid for most engineering applications.

The three most common types of tests to determine the shear strength of soils are the direct shear test, the triaxial test, and the unconfined compression test. The unconfined compression test is actually a special case of the undrained triaxial test in which the confining pressure is zero.

2.2.3. a <u>Unconfined Compressive Strength (q_u).</u> The Road Research Laboratory (89) described a curvilinear relationship between unconfined compressive strength (q_u) and moisture content (w_n) for a heavy clay in situ. This relationship was characterized by a large increase in unconfined compressive strength with decreasing moisture content below 30 percent.

Sherif and Burrous (100) investigated the effect of temperature on unconfined compressive strength of kaolinite. It was assumed that a change in temperature would cause a change in void ratio or a change in effective stress (or a combination of both) for a saturated compacted

clay. Thus, a change in temperature could cause a change in strength. Their test results showed that: (1) as the moisture content increases, the unconfined compressive strength decreases, and (2) at a given moisture content as the temperature increases, the unconfined compressive strength decreases. The temperature effects on the undrained strength for compacted soils were further investigated and verified by Highter (39).

Sahasi (94) studied the correlation of unconfined compressive strength of soil with size of the test specimen. The Black Cotton soil (CH) at PUNE in India was used as the test material. His conclusions include the following: (1) there exists a unique relationship between the unconfined compressive strength and the moisture content, (2) the relation between unconfined compressive strength, q_u in kg/cm^2, moisture content, w_n in percentage, and specimen diameter, D in cm., may be expressed empirically as

$$\text{Log } q_u = 0.52\ (0.502 - w_n - 0.163 \log D)$$

In his study on compacted clay of St. Croix, Indiana, Weitzel (120) found that at zero confining pressure the model for strength prediction dry-of-optimum is:

$$\hat{q}_c = -1784.8 + 3.1\ \rho_d \sqrt{S_i}/w$$

where $\hat{q}_c$ is the estimated compressive strength, kN/m^2,

ρ_d is the dry density, kg/m^3,

S_i is the initial degree of saturation, %,

and w is the water content, %.

The model for strength prediction wet-of-optimum is:

$$\log (\hat{q}_c) = 1.70/e_i$$

where e_i is the initial void ratio.

Peters and Lamb (82) presented an empirical relationship between liquidity index (LI) and unconfined compressive strength (q_u) for the soils in western Canada. They showed that as the liquidity index decreases, the unconfined compressive strength increases.

Peck et al (80) suggested a relationship between the qualitative terms describing consistency, along with field identification of clays, and the quantitative values of the unconfined compressive strength. The National Research Council of Canada (74) also suggested a similar relationship for the rough estimate of the undrained shear strength (half of the unconfined compressive strength) for clay soils.

The unconfined compressive strength (q_u) has been correlated with the standard penetration test (SPT), i.e., the number of blows (N-values) for one foot penetration. Some of these relationships are presented as follows. In his study on Tokyo subsoils, Ohsaki (77) found that the

relationship between q_u of clayey samples and their N-values was:

$$q_u(kg/cm^2) = 0.4 + \frac{N}{20}$$

However, an examination of the data shows much scatter. This expression, therefore, is qualitative rather than quantitative.

The U.S. Navy (118) recommended that for clayey silts, CL clays, or varved clays and silts,

$q_u(Tsf) = 0.15N$, and for CH clays

$q_u(Tsf) = 0.20N$

Terzaghi and Peck (113) also suggested relations among consistency of clay described in qualitative terms, N-values, and unconfined compressive strength. Sanglerat (95) recommended the following expressions:

for clay, $q_u\ (Tsf) = \frac{N}{4}$

for silty clay, $q_u\ (Tsf) = \frac{N}{5}$

and for silty, sandy soil, $q_u\ (Tsf) = \frac{N}{7.5}$.

2.2.3. b <u>Undrained Shear Strength (c_u).</u> The undrained shear strength (c_u) under $\phi = 0$ conditions is, in terms of unconfined compressive strength (q_u),

$$c_u = \frac{1}{2} q_u$$

The undrained shear strength is also determined, in situ, by means of the vane shear test and other less common procedures.

Due to the fact that the undrained shear strength (c_u) of normally consolidated clay soils is almost directly

proportional to the effective overburden pressure (p_o), Skempton (105) suggested the following empirical relationship between the plastic index and the ratio c_u/p_o:

$$c_u/p_o = 0.11 + 0.0037\ I_p$$

For saturated sand Bishop and Eldin (9) found that as the initial porosity (n) increases, the ratio c_u/p_o decreases. And for a given initial porosity the higher the effective overburden pressure, the lower the ratio c_u/p_o.

Bjerrum and Simons (10) stated that the well-known Skempton-Bjerrum correlation of increasing ratio c_u/p_o with increasing plastic index applied only to normally consolidated marine clays of Norway. They also defined c_u/p_o as a function of the liquidity index (LI) for some Norwegian clays, to illustrate the existence of an unstable structure for quick clays with ratio c_u/p_o less than 0.15.

The U.S. Navy (118) suggested correlations of the unconfined undrained shear strength (or cohesion intercept in saturated state) with Unified classification. Of course, the unconfined undrained shear strength (c_u) is zero for granular soils and is as high as 420 to 460 psf for silty clay or clayey silt soils.

In their study of the geotechnical characteristics of till deposits of the Quaternary Period in the Edmonton area, Alberta, Canada, May and Thomson (61) found a distinct trend for higher undrained shear strengths with lower moisture contents.

Using the assumption that the mean value of undrained shear strength is about 1.70 kN/m^2 (from the results of Youssef et al (124)) for a variety of remolded Egyptian soils, Wroth and Wood (123) developed the following relationship

$$c_u = 170 \text{ EXP } (-4.6 \text{ LI}) \text{ kN/m}^2$$

2.2.3. c <u>Standard Penetration Resistance (N) and Strength Angle (ϕ')</u> (1) <u>Standard Penetration Resistance (N-value).</u> The standard penetration resistance (N-value) varies with the relative density or relative consistency of soils, and is usually evaluated in terms of effective overburden pressure (p_o) and relative dry density (D_r) for cohesionless cases. Sanglerat (95) has documented many equations and charts from all over the world. One such equation was established by Bazaraa (7) in the form,

$$N = 20\, D_r^{\,2}\, (1 + 2P_o) \text{ for } p_o \leq 1.5 \text{ kips/ft}^2,$$

and $$N = 20\, D_r^{\,2}\, (3.25 + 0.5\, P_o) \text{ for } p_o \geq 1.5 \text{ kips/ft}^2.$$

The U.S. Navy (118) developed a correlation between N and relative dry density (D_r) which included the effective overburden pressure (P_o). Peck et al (80) related N and the relative density in qualitative terms for sands.

(2) <u>Strength angle (ϕ').</u> Ohsaki (77) suggested the following relationship between ϕ' and N for the sandy subsoils of Tokyo

$$\phi' = \sqrt{20\, N + 15}.$$

However, an examination of the plot of ϕ' versus N shows

that the data is scattered. For sand, Caquot and Kerisel (15) found a relationship

$$\tan \phi' = \frac{A}{e}$$

where e is void ratio, and A a constant assumed to be 0.55. Later (16), they proposed that under an overburden pressure of 1 bar ($\simeq$ 1 ton/sq. ft.):

for silts: $A = 0.4$

and for sands: $0.45 < A < 0.55$.

Meyerhof (65) proposed that the relationship between ϕ' and D_r be expressed as

$$\phi' = 25 + 0.15\ D_r,$$

for granular soil containing more than 5% fine sand and silt, and

$$\phi' = 30 + 0.15\ D_r$$

for granular soil containing less than 5% fine sand and silt. D_r is relative dry density expressed as a percentage.

Bjerrum and Simons (10) presented the relationship between strength angle and plastic index for different clays. However, Terzaghi and Peck (113) questioned the general validity of this relationship, citing test data from Mexico City soils. Also, in their study of the strength characteristics of Kuttanad clays in India, Narain and Ramanathan (72) found that there was no definite correlation between strength angle and plastic index. However, the Canadian PFRA (82, 88) found that

as the liquid limit increased, the strength angle decreased for undisturbed clays in western Canada.

2.3. Discussion

Review of the large number of empirical relationships documented in the geotechnical engineering literature leads to two principal conclusions:

(1) The values of soil parameters are expressed as means.

(2) The functional relationships among soil characteristics are established with data from a certain region or pooled data from several regions. The regional effects on relationships are not commonly investigated or compared.

Jenny (45) gave a generalized equation form of soil-forming factors as:

$$s = f(cl, o, r, p, t....)$$

where s is soil, cl climate, o organisms, r topography, p parent material, and t time. These factors have imposed a random pattern of variation onto an overall trend of soil formation. The resulting soil characteristics can be regarded as being controlled by a random process. The random process of soil formation explains the great variation often encountered for a given soil parameter. Therefore, it seems more reasonable to define and use the median, rather than mean, for soil parameter values.

The regional effects on functional relationships among soil characteristics should be investigated by using the techniques of qualitative variables as regressors associated with analysis of variance. The details of these procedures are discussed and illustrated in Section 3.3.2.a.

CHAPTER 3. THE DEVELOPMENT OF THE GEOTECHNICAL DATA BANK FOR THE STATE OF INDIANA

3.1. General

Goldberg (35) initiated the development of the Indiana geotechnical data bank. A total of 2508 data sets were collected and subjected to statistical analyses in an initial research phase. An additional 6934 data sets were added to the bank in the subsequent year and subjected to more detailed statistical analyses. As of January 1980, the Indiana geotechnical data bank contained 9442 data sets. The distribution of data sets throughout the State is shown in Figure 3-1.

In the following, the source of data, structure of data, and methods leading to the building of mathematical models are discussed. Results are given in Appendix A.

3.2. Source and Structure of Data

Both geotechnical and pedological soils information was collected. The geotechnical information was taken from the subsurface investigation reports of various geotechnical projects previously conducted in the State of Indiana, and the pedological soils information was from recent agriculture Soil Survey Manuals (105) and General Soil Maps

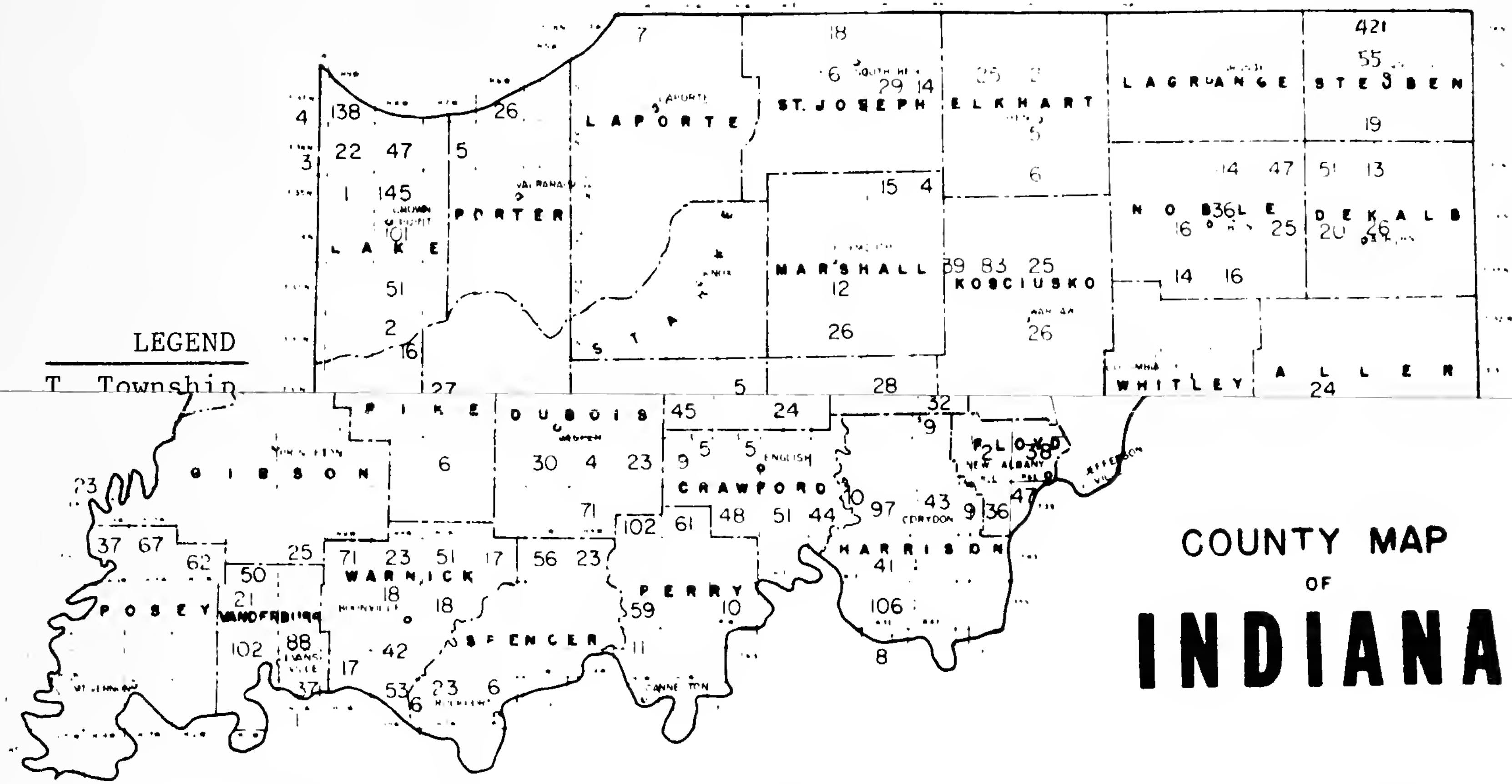

Figure 3-1. Soil Data Distribution Map, Indiana Geotechnical Data Bank

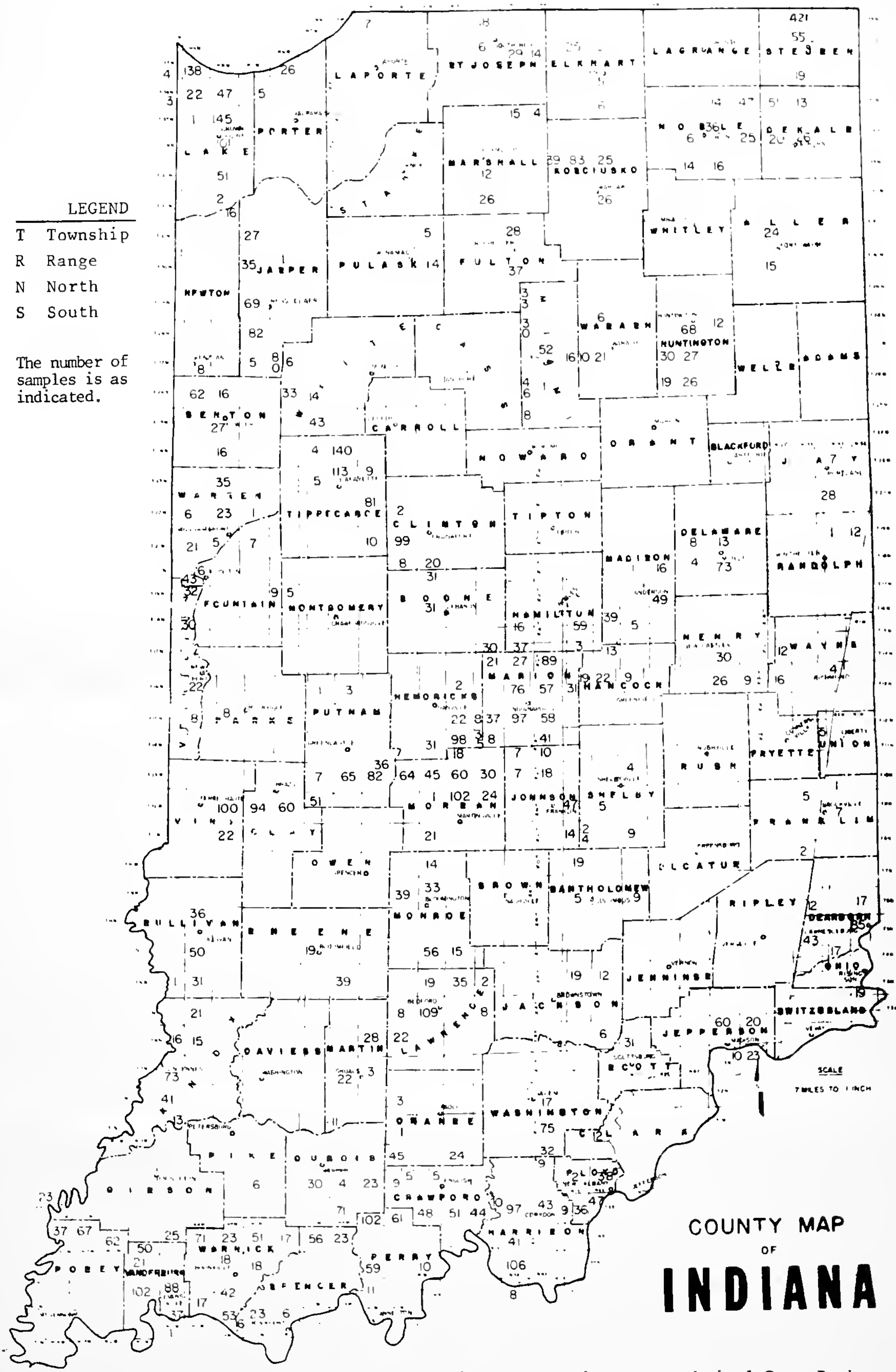

Figure 3-1. Soil Data Distribution Map, Indiana Geotechnical Data Bank

(32). For details refer to Goldberg (35).

The Data Input Form (DIF) of Figure 3-2 was developed to record the information. Not all items were available for all samples, but the listing included the following:

1. Project identification,
    a. project number
    b. contract number
    c. road number
    d. data collection agency
2. Sample location,
    a. county
    b. highway district
    c. township
    d. range
    e. section
    f. line number
    g. station number
    h. offset and the left or right direction from the center line.
3. Sample identification,
    a. boring number
    b. laboratory number
    c. sampling procedure
4. Date the sample taken from the hole,
5. Physiographic region,

ECORDED BY: DATE:

SEQNUM: HECKED BY: DATE:

CARD	C O		HOLE NUMBER					SAMP NO		MBER	SUF		BORING NUMBER									SOIL ASSOC.				REPEAT
1	2	3	4	5	6	7	8	9	10	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80
1																										

CARD	ID NO.									DEPTH BOTTOM				SPT				PHYSIO				SOIL SERIES				
1	2	3	4	5	6	7	8	9	10	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80
2																										

CARD	ID NO.									E		3/4" SIEVE					1/2" SIEVE					3/8" SIEVE				
1	2	3	4	5	6	7	8	9	10	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80
3																										

CARD	ID NO.									L			PI					SL				LOSS ON IGNIT %				
1	2	3	4	5	6	7	8	9	10	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80
4																										

CARD	ID NO.										CBR SOAKED 100% MAX DD				CBR SOAKED 95% MAX DD				UNCONFINED COMPRESSIVE STRENGTH TSF				AASHTO		UNIF	
1	2	3	4	5	6	7	8	9	10	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80
5																										

CARD	ID NO.									TEST																
											P_f TSF				C_c				C_r				C_v FT ^{2}/MTH			
1	2	3	4	5	6	7	8	9	10	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80
6																										

COMMENTS:

Figure 3-2 Data i

DATA INPUT FORM

COMPUTERIZED SOIL DATA FOR THE STATE OF INDIANA

SEQNUM: ____________

RECORDED BY: ____________ DATE: ____________

CHECKED BY: ____________ DATE: ____________

CARD	CO	HOLE NUMBER	SAMP NO	DIST	YEAR	MONTH	DAY	TOWN	DIR	RANGE	DIR	SECT	PROJECT NUMBER: PRE	NO.	PAREN	MILE	CONTRACT NUMBER: PRE	NO.	ROAD NUMBER: PRE	NO	SUF	BORING NUMBER	SOIL ASSOC.	REPEAT
1																								

CARD	ID NO.	STATION NUMBER	OFFSET	DIR	LINE NUMBER	SOURCE	SAMPLE	TYPE	LAB NUMBER	GROUND SURFACE ELEVATION	DEPTH TOP	DEPTH BOTTOM	SPT	PHYSIO	SOIL SERIES
2															

CARD	ID NO.	PARENT	HOR	SLOPE	ERO	BEDRK SOIL SURVEY	BEDRK BORING LOG	WATERS	WATER COMPLETION	WATER: FINAL OR 24 HOURS	DRAIN	PERMEA	FLOOD	FROST	SHRINK	PH	1-1/2" SIEVE	1" SIEVE	3/4" SIEVE	1/2" SIEVE	3/8" SIEVE
3																					

CARD	ID NO.	NO 4 SIEVE	NO 10 SIEVE	NO 40 SIEVE	NO 200 SIEVE	NO 270 SIEVE	% SAND	% SILT	% CLAY	% COLLOIDS	LL	PL	PI	SL	LOSS ON IGNIT %
4															

CARD	ID NO.	NATMC %	NATWD PCF	NATDD PCF	SPECIFIC GRAVITY	TEXTURE	ORG	COLOR	COMPACT	EFFORT	MAXDD PCF	MAXWD PCF	OPTIM MOIST CONTENT %	CBR UNSOAK 100% MAX DD	CBR UNSOAK 95% MAX DD	CBR SOAKED 100% MAX DD	CBR SOAKED 95% MAX DD	UNCONFINED COMPRESSIVE STRENGTH TSF	AASHTO	UNIF
5																				

CARD	ID NO.	FAILURE STRAIN %	TYPE	STRENGTH TESTS: STRENGTH TSF	STRAIN %	CONFINING PRESSURE PSI	COHESION TSF	$\phi°$	PORE PRESSURE PSI	σ_1 TSF	CONSOLIDATION TEST: e_o	e_f	S_o %	S_f %	P_o TSF	P_f TSF	C_c	C_r	C_v FT^2/MTH
6																			

COMMENTS

Figure 3-2 Data input form (DIF)

6. Parent material from which the soil has been derived,
7. Ground surface elevation,
8. Depth from which the sample has been removed,
9. Depth to the bedrock,
10. Depth to groundwater,
11. Standard penetration resistance (SPT),
12. Pedological soils information,
    a. soil association name
    b. soil series name
    c. horizon
    d. slope (topographic) class
    e. erosion class
    f. natural soil drainage class
    g. generalized permeability
    h. generalized flooding potential
    i. generalized frost heave susceptibility
    j. generalized shrink - swell potential
    k. generalized pH
13. Gradational characteristics based on standard sieve sizes and hydrometer analysis,
14. Atterberg limits,
15. Visual textual classifications,
16. Color based on moist conditions,
17. Organic content (loss on ignition),

18. In-situ moisture content,
19. In-situ dry and wet densities,
20. Specific gravity,
21. Compaction test results,
22. California bearing ratio (CBR),
23. Unconfined compressive strength and failure strain,
24. Strength data from triaxial and direct shear tests,
25. Consolidation test results.

Details of these listing and their corresponding coding systems are described in Goldberg (35). Computer programs to utilize the information from (13) to (16) to classify the samples by the AASHTO and Unified soil classification systems, to detect certain input errors, and to correlate a soil association with its most probable corresponding parent material are listed in Appendix B-II.

Those programs shown in Appendix B-II were written in the SPSS (Statistical Package for the Social Sciences) language available in the PUCC (Purdue University Computing Center) for CDC 6500 and 6600 systems. The SPSS language, like any other recent statistical language, is a "conversational" statistical analysis software. It also has features of data manipulation, data transformation, file definition, and file creation. For details refer to

Nie et al (75). This study has relied heavily upon this language. However, any computer language is merely an access to an optimum use of the computer hardware. Therefore, the logical sequences of a program are more important than the program itself. The logic of these programs, as listed in Appendix B-II, should be regarded as the guideline for the ready conversion to any other computer language. And the methods of analyses as described in the following are designed and written for adaptation to any language.

3.3. Methods of Analysis

Two types of prediction equations were used in this work: (1) median models, and (2) regression models.

3.3.1.a. Median Model

To numerically define the variability of selected soil characteristics, the frequency distributions of these characteristics are examined and described. One way to describe the sample distribution is to use the conventional constant mean model (33), which is based upon the assumption of normality of the population distribution. This model is characterized by the mean and the standard deviation of the sample distribution. It is shown (51) that in most cases this model is also effective for moderately anormal distributions of a large sample. In the case of small samples, however, this model may not give

accurate approximations. The non-parametric or distribution-free methods are the preferred techniques of inference for non-normal population (Snedecor and Cochran (106). These methods make a minimum of assumptions regarding the sample distribution and are generally appropriate for any form of the distribution. They are of high efficiency, relative to classical techniques, under the assumption of normality, and often of higher efficiency in other situations (51). For further details refer to Hollander et al (42) and Snedecor (106). In this study the samples distributions were sometimes normal, but frequently they were skewed or bimodal. The median model was used to characterize the sample distribution.

To estimate the median from a sample the observations are arranged in increasing order. When the sample values are arranged in this way, they are often called the 1st, 2nd, 3rd,...order statistics. In general, if n is odd, the median is the order statistic whose number is $\frac{n+1}{2}$. With n even, the median is defined as the average of the order statistics whose numbers are $\frac{n}{2}$ and $\frac{n+2}{2}$. Like the mean, the median is a measure of the middle of a distribution. If the distribution is symmetrical about its mean, the mean and the median coincide. With highly skewed distributions the median is preferred, since it seems to better represent the concept of an average than the mean.

The calculations of the median and other non-parametric estimates from a large sample are illustrated in the following example with the data from the current Indiana geotechnical data bank. The example is for 43 observations of shrinkage limits for the soil association Berks-Gilpin-Weikert with the depth less than 5 feet. The observations are arranged in an increasing order and the proper intervals for the distribution are determined. The results are shown in Table 3-1. with the data from Indiana Geotechnical Data Bank.

Table 3-1. Distribution of Shrinkage Limits for the Soil Association Berks-Gilpin-Weikert with the Depth Less than 5 Feet

Shrinkage Limit (in %)	12.00-15.00	15.00-18.00	18.00-21.00	21.00-24.00	24.00-27.00	27.00-30.00	30.00-33.00
Frequency	8	5	6	5	9	7	3
Cumulative Frequency	8	13	19	24	33	40	43

There are two modes in the distribution. The first mode is in the class from 12.00 to 15.00, and the second mode in the class from 24.00 to 27.00. This bimodal feature emphasizes the non-normality of the distribution.

Since the number of observation (n) is equal to 43, the sample median is the order statistic that is 22nd in the array. It is clear that the median is in the class of 21.00 - 24.00. The median is found by interpolation with the assumption of uniform distribution of observations in this class. The general equation is (106)

$$t_{.50} = x_L + \frac{gI}{f} \tag{3.1}$$

where x_L = value of x at lower limit of the class containing the median (21.00 in this case),

g = order statistic number of the median minus cumulative frequency up to the upper limit of previous class (22 - 19 = 3),

I = class interval (3%),

f = frequency in class containing the median (5).

This gives

$$t_{.50} = \text{median} = 21.00 + \frac{(22.00 - 19.00)\ (3)}{5}$$

$$= 22.80\%$$

There is a simple method of calculating confidence intervals for the population median that is valid for any continuous distribution. Two of the order statistics serve as the upper and lower confidence interval limits. These are the order statistics whose numbers are, approximately (67)

$$\frac{n + 1}{2} \pm \frac{z\sqrt{n}}{2}\ , \tag{3.2}$$

where z is the normal deviate corresponding to the desired confidence probability. Consider a 95% confidence probability, $z \simeq 2$. In this example, these numbers are $\frac{43 + 1}{2} \pm \frac{2\sqrt{43}}{2}$ = 15 and 28. The 95% confidence intervals of the median are the shrinkage limits corresponding to the 15th and 28th order statistics. The actual shrinkage limits are found by adapting equation 3.1 for the median.

For the 15th, shrinkage limit = 18 + (15 - 13) (3)/5
= 19.20%

For the 28th, shrinkage limit = 24 + (28 - 24) (3)/9
= 25.33%

The population median is between 19.20% and 25.33% except for unusual values that occur about once in twenty trials.

In any continuous frequency distribution the p-th percentile is estimated by the order statistic whose number is (n + 1)p/100. For the 43 shrinkage limits, the 25th percentile is estimated by an order statistic whose number is $i_{.25} = (43 + 1)25/100 = 11$. Again by using equation 3.1, the shrinkage limit corresponding to the 25th order statistic is

$$t_{.25} = 15 + (11 - 8)(3)/5 = 16.80$$

In the same way, the number of the order statistic 75th percentile ($t_{.75}$) is $i_{.75} = 33$; and $t_{.75} = 27.00$. The interquartile range (IR) is defined as the difference between the 75th and the 25th percentiles. In this case, IR = 27.00 - 16.80 = 10.20. The IR is used as a measure of population

variability rather than standard deviation. The median, confidence interval of median, percentiles, and inter-quartile range are known as the non-parametric estimates of a sample distribution.

In this case, the mean of the distribution is 22.4 and the standard deviation is 5.632. The 95% confidence interval of the mean is, approximately,

$$22.4 \pm \frac{2(5.632)}{\sqrt{43}} = 20.68 \sim 24.12$$

Let us consider another example. Table 3-2 shows 35 observations of shrinkage limits for the soil association Cincinnati-Rossmoyne-Hickory with the depth less than 4 feet. By observation, the distribution is skewed.

Table 3-2. Distribution of Shrinkage Limits for the Soil Association Cincinnati-Rossmoyne-Hickory with the Depth Less than 4 Feet

Shrinkage Limit (%)	9.00-12.00	12.00-15.00	15.00-18.00	18.00-21.00	21.00-24.00	24.00-27.00	27.00-30.00	30.00-33.00
Frequency	2	6	13	8	3	1	1	1
Cumulative Frequency	2	8	21	29	32	33	34	35

The same procedures were employed to attain the non-parametric estimates of the sample distribution. The results are as follows.

The median, $t_{.50}$, is equal to 17.31 and the 95% confidence interval of the median is 15.92 - 19.13. The 25th and 75th percentiles are 15.23 and 20.25, respectively. Accordingly the IR = 5.02. The mean of this distribution is 18.37 and the standard deviation is 4.35. Therefore, the 95% confidence of the mean is, approximately,

$$18.37 \pm \frac{2(4.35)}{\sqrt{35}} = 16.90 \sim 19.84$$

For a final example, Table 3-3 shows 38 observations of natural moisture contents for the soil association Huntington-Wheeling-Markham with depths between 5 and 10 feet. This distribution is seen to be symmetrical.

Table 3-3. Distribution of Natural Moisture Contents for the Soil Association Huntington-Wheeling-Markham with Depths between 5 and 10 Feet

Natural Moisture Content (%)	15.00-20.00	20.00-25.00	25.00-30.00	30.00-35.00	35.00-40.00
Frequency	1	7	20	8	2
Cumulative Frequency	1	8	28	36	38

The same procedures were employed. With the following results. The median is $t_{.50}$ = 27.87 and the 95% confidence interval of the median is 26.25 ~ 29.50. The 25th and 75th percentiles are 25.50 and 30.62, respectively.

The interquartile range (IR) = 5.12. The mean of this distribution is 27.77 and the standard deviation is 3.942. The 95% confidence interval of the mean is, approximately,

$$27.77 \pm \frac{2(3.942)}{\sqrt{38}} = 26.49 \sim 29.05$$

A summary of results obtained above is shown in Table 3-4. For a bimodal distribution, the interquartile range is greater than the standard deviation. This is expected due to the bimodal feature. In the case of a skewed distribution the mean is greater than the median because of the skewness. As for symmetrical distributions, since the interquartile range is greater than the standard deviation, the non-parametric method is less efficient than the classical techniques based upon the assumption of normality. However, the non-parametric method still yields reasonable results.

3.3.1.b. Applications of the Median Model

The methods and procedures illustrated in Section 3.3.1.a. were used to describe the following characteristics:

(1) topographic characteristics, viz., ground elevation, ground water elevation, and water depth with relation to ground elevation;

(2) the relationship between the remolded soil characteristics and AASHTO classification within a physiographic region;

Table 3-4 A Summary of Results from Table 3-1 to Table 3-3

Source	Median ($t_{.50}$)	Mean ($\hat{\mu}$)	95% Confidence Interval of Median	95% Confidence Interval of Median	Interquartile Range (IR)	Standard Deviation (s.d.)
Table 3-1 (Bimodal Distribution)	22.80	22.40	19.20 ~ 25.33	20.68 ~ 24.12	10.20	5.63
Table 3-2 (Skewed Distribution)	17.31	18.37	15.92 ~ 19.13	16.90 ~ 19.83	5.02	4.35
Table 3-3 (Symmetrical Distribution)	27.87	27.77	26.25 ~ 29.50	26.49 ~ 29.05	5.12	3.94

and

(3) statistical soil profiles.

Each item is described as follows.

(1) Topographic characteristics:

The topographic characteristics were examined within physiographic regions. The ground water depth with relation to ground elevation was taken directly from the WATER FINAL as recorded on the DIF. The ground water elevation was defined as the difference between the ground elevation and the WATER FINAL.* The procedures of analyses are outlined as follows under the assumption of a uniform distribution of the observations of these topographic characteristics throughout a physiographic region:

(a) Examine the sample distributions of these topographic characteristics for a desired physiographic region, and

(b) Apply the methods discussed in Section 3.3.1.a. to describe these sample distributions.

The results are shown in Appendix A-I.

(2) Relationships between the remolded soil characteristics and AASHTO classification within a physiographic region:

There exist many relationships between the soil classification, notably the AASHTO, and the characteristics of remolded soils, as discussed in Chapter 2. To develop these kinds of relationships on a regional basis, the following procedures were used.

*WATER FINAL is defined as the final or 24 hour reading, whichever is reported in the drill logs, of the depth to water.

(a) Examine the distribution of AASHTO classification units within a given physiographic region (Appendix A-II) and select three or four most probable AASHTO classification units as the representative soil groups in the region.

(b) Examine the distributions of both visual texture and Unified classification units for each of the selected AASHTO classification units within the specific region. Select the most probable texture and Unified classification units as the representatives for soil identification and correlation.

(c) Apply the methods described in Section 3.3.1.a. to obtain the estimates of samples distributions of the following remolded soil characteristics: natural dry density (ρ_d), specific gravity (G), shrinkage limit (SL), maximum dry density ($\rho_{d\ max}$), optimum moisture content (OMC), CBR value at 100% maximum dry density (CBR S01), and CBR value at 95% maximum dry density (CBR S02) for each AASHTO classification unit selected above. The results are shown in Appendix A-II.

(3) Statistical soil profiles

An attempt has been made to generate soil profiles with statistical bases. As the statistical reasoning is based on the characteristics of an aggregate of sample observations, any discontinuity in the sample distribution must be eliminated. The statistical soil profiles were

generated according to the pedological soil associations, because these are the only soil grouping units which are reasonably large and grossly homogeneous. The procedures of generating a statistical soil profile for a given soil association are illustrated below.

Sufficient data were available for the soil association Wellston-Zanesville-Berks (Table 3-18). The items of general description and parent material in Table 3.18 were extracted from the "General Soils Maps" (32) and the "Key to Soils of Indiana" (30). Examinations of sample distributions of AASHTO classification units, counties, and physiographic regions for this soil association as shown in Tables 3-5 through 3-7 indicate that: (i) the A-4, A-6, and A-7-6 soils are more dominate than the others, and, therefore, are selected as the soil representatives for further layer divisions; and (ii) this kind of soil is mainly distributed in Crawford, Dubois, and Perry counties within the Crawford Upland physiographic regions.

Tables 3-8 to 3-10 show the distributions of topographic characteristics. Only those ground elevations which have corresponding WATER FINAL records were selected. The methods presented in Section 3.3.1.a. were employed to describe these samples distributions.

The 90th percentile of the DEPTH T (as recorded on the DIF) for each of the AASHTO units selected was

Table 3-5 Distribution of AASHTO Units for Soil Association Wellston-Zanesville-Berks

AASHTO Unit	Frequency	Relative Frequency (%)	Cumulative Frequency (%)
Unknown	7	2.6	2.6
A-1-B	2	0.8	3.4
A-2-4	19	7.1	10.5
A-2-6	3	1.1	11.7
A-4	73	27.4	39.1
A-6	77	28.9	68.0
A-7-5	12	4.5	72.6
A-7-6	73	27.4	100.0
Total	266	100.0	100.0

Table 3-6 Distribution of Counties for Soil Association Wellston-Zanesville-Berks

County	Frequency	Relative Frequency (%)	Cumulative Frequency (%)
Crawford	110	41.4	41.4
Dubois	67	25.2	66.6
Harrison	7	2.6	69.2
Orange	17	6.4	75.6
Perry	65	24.4	100.0
Total	266	100.0	100.0

Table 3-7 Distribution of Physiographic Regions for Soil Association Wellston-Zanesville-Berks

Physiographic Region	Frequency	Relative Frequency (%)	Cumulative Frequency (%)
Crawford Upland	252	94.7	94.7
Wabash Lowland	14	5.3	100.0
Total	266	100.0	100.0

Table 3-8 Distribution of Ground Elevations Which Have Corresponding WATER FINAL Records for Soil Association Wellston-Zanesville-Berks

Class Limit (ft.)	420-470	470-520	520-570	570-620	620-670	670-720	720-770	770-820
Frequency	20	8	11	6	8	2	1	3
Cumulative Frequency	20	28	39	45	53	55	56	59

Table 3-9 Distribution of WATER FINAL for Soil Association Wellston-Zanesville-Berks

Class Limit (ft.)	1-5	5-9	9-13	13-17	17-21	21-25	25-29	29-33	33-37
Frequency	27	14	6	6	3	1	1	0	1

Table 3-9 (continued)

Cumulative Frequency	27	41	47	53	56	57	58	58	59

Table 3-10 Distribution of Ground Water Elevation (Elevation of Ground Surface - WATER FINAL) for Soil Association Wellston-Zanesville-Berks

Class Limit (ft.)	415-465	465-515	515-565	565-615	615-665	665-715	715-765	765-815
Frequency	19	10	11	5	8	2	1	3
Cumulative Frequency	19	29	40	45	53	55	56	59

Table 3-11 Distribution of DEPTH T of A-4 Soils for Soil Association Wellston-Zanesville-Berks

Class Limit (ft.)	0-1	1-2	2-3	3-4	4-5	5-6	6-7	9-10	10-11	11-12	14-15	15-16	18-19	20-21	22-23
Frequency	16	7	7	6	10	1	5	1	1	2	4	2	5	3	3
Cumulative Frequency	16	23	30	36	46	47	52	53	54	56	60	52	67	70	73

defined as its upper depth boundary in the profile, and the 90th percentile of DEPTH B (as recorded on the DIF), as its lower depth boundary. This was done to eliminate the extremes of DEPTH T and DEPTH B and, hopefully, to minimize the discontinuity of soil sample versus depth distributions. For example, as shown in Table 3-11, the number in order statistics corresponding to the 90th percentile of the distribution of DEPTH T of A-4 soils is (73 + 1)90% = 67. Therefore, the 90th percentile is equal to 19 feet, which gives the upper depth boundary of A-4 soil. In the same manner the 90th percentile of the distribution of DEPTH B of A-4 soil is 21 feet, which defines the lower depth boundary (Table 3-12). Using the sample distributions described in Table 3-13 to 3-16, and applying the same procedures, the upper depth boundary is 15 feet and the lower depth boundary is 16 feet for A-6 soil. The upper depth boundary is 12 feet and the lower depth boundary 17.50 feet for A-7-6 soil. The results are shown in Figure 3-3(a). Table 3-17 shows the distribution of all available AASHTO units versus DEPTH T.

An examination of Figure 3-3(a) and Table 3-17 indicates that A-4, A-6, and A-7-6 soils are more dominate from the surface to a depth about 12 feet. The A-6 soil is more dominate from 12 feet to approximately 17 feet. Thus a three layer system is drawn for this soil profile.

Table 3-12 Distribution of DEPTH B of A-4 Soils for Soil Association Wellston-Zanesville-Berks

Class Limit (ft.)	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10
Frequency	1	9	5	10	4	6	12	2	1	2
Cumulative Frequency	1	10	15	25	29	35	47	49	50	52

Table 3-12 (continued)

Class Limit (ft.)	11-12	12-13	13-14	15-16	16-17	17-18	19-20	20-21	22-23	24-25
Frequency	1	1	2	1	3	2	4	1	3	3
Cumulative Frequency	53	54	56	57	60	62	66	67	70	73

Table 3-13 Distribution of DEPTH T of A-6 Soils for Soil Association Wellston-Zanesville-Berks

Class Limit (ft.)	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12
Frequency	25	5	6	4	9	2	2	1	2	3	1	4

Table 3-13 (continued)

Cumulative Frequency	25	30	36	40	48	51	53	54	56	59	60	64

Table 3-13 (continued)

Class Limit (ft.)	12 -13	13 -14	14 -15	15 -16	16 -17	18 -19
Frequency	3	2	1	2	3	2
Cumulative Frequency	67	69	70	72	75	77

Table 3-14 Distribution of DEPTH B of A-6 Soils for Soil Association Wellston-Zanesville-Berks

Class Limit (ft.)	0 -1	1 -2	2 -3	3 -4	4 -5	5 -6	6 -7	7 -8	8 -9	9 -10
Frequency	2	13	4	7	9	4	8	5	2	1
Cumulative Frequency	2	15	19	26	35	39	47	52	54	55

Table 3-14 (continued)

Class Limit (ft.)	10 -11	11 -12	12 -13	13 -14	14 -15	15 -16	17 -18	18 -19	19 -20	20 -21
Frequency	4	1	1	4	3	2	2	3	1	1
Cumulative Frequency	59	60	61	65	68	70	72	75	76	77

Table 3-15 Distribution of DEPTH T of A-7-6 Soils for Soil Association Wellston-Zanesville-Berks

Class Limit (ft.)	0 -1	1 -2	2 -3	3 -4	4 -5	5 -6	6 -7	7 -8	8 -9	9 -10
Frequency	13	7	7	7	8	5	6	1	4	4
Cumulative Frequency	13	20	27	34	42	47	53	54	58	62

Table 3-15 (continued)

Class Limit (ft.)	10 -11	11 -12	13 -14	15 -16	18 -19	19 -20	20 -21
Frequency	2	3	1	2	1	1	1
Cumulative Frequency	64	67	68	70	71	72	73

Table 3-16 Distribution of DEPTH B of A-7-6 Soils for Soil Association Wellston-Zanesville-Berks

Class Limit (ft.)	1 -2	2 -3	3 -4	4 -5	5 -6	6 -7	7 -8	8 -9	9 -10	10 -11
Frequency	10	2	9	8	7	4	6	5	4	3
Cumulative Frequency	10	12	21	29	36	40	46	51	55	58

Table 3-16 (continued)

Class Limit (ft.)	11 -12	12 -13	13 -14	15 -16	17 -18	19 -20	22 -23	23 -24	49 -50
Frequency	3	2	2	1	3	1	1	1	1
Cumulative Frequency	61	63	65	66	69	70	71	72	73

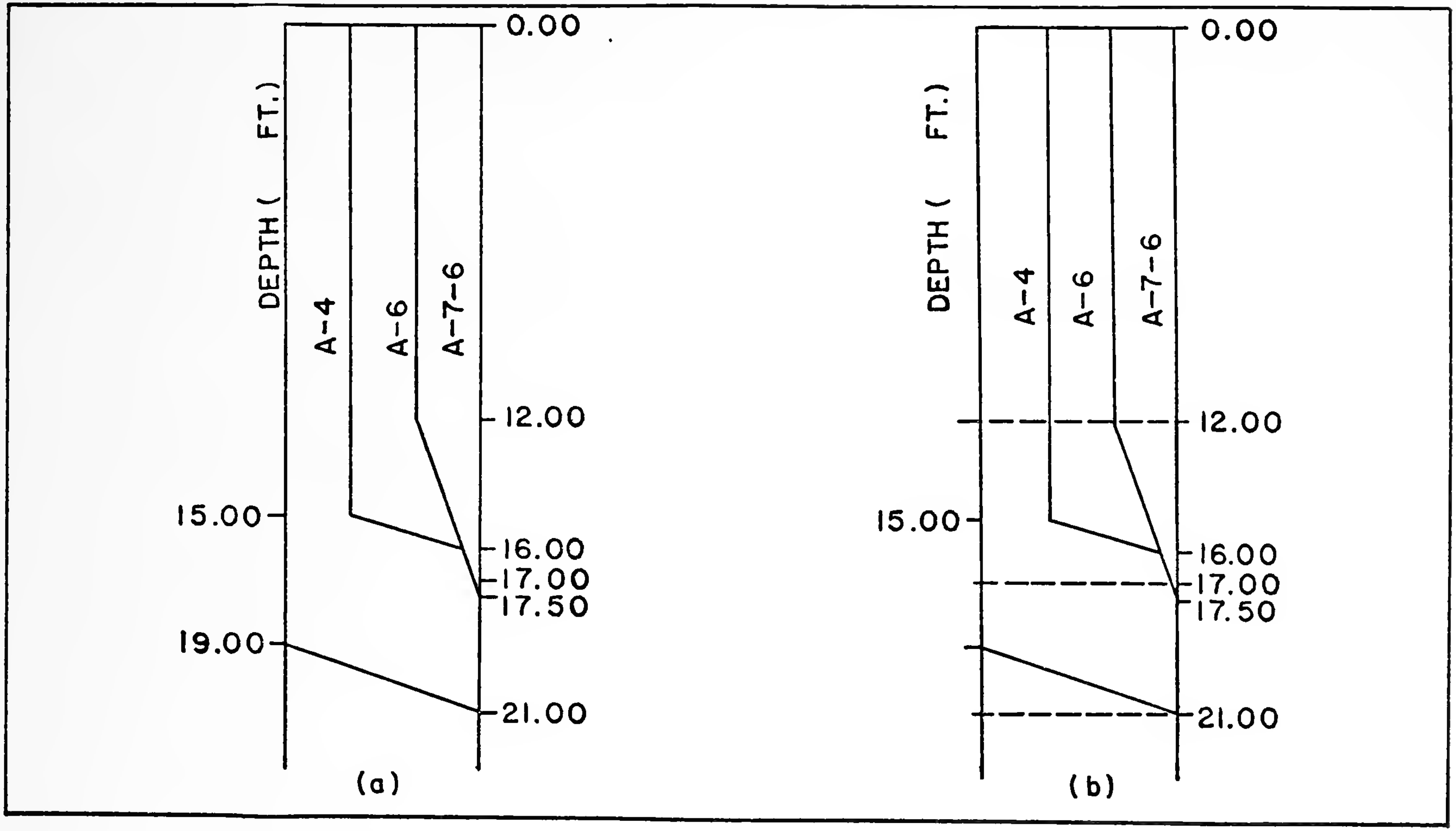

Figure 3-3 Construction of a Soil Layer System

Table 3-17. AASHTO Units vs. DEPTH T for Soil Association Wellston-Zanesville-Berks

AASHTO / DEPTH T (ft)	A-1-A	A-1-B	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-4	A-5	A-6	A-7-5	A-7-6	A-8
0		1	5					16		25	1	13	
1			1					7		5		7	
2			2					7		6		7	
3		1	2					6		4		7	
4			2		2			10		9	1	8	
5			1					1		2	2	5	
6			1					5		2	1	6	
7										1		1	
8			2							2		4	
9			2					1		3	2	4	
10								1		1		2	
11								2		4		3	
12					1					3	1		
13										2	2	1	
14								4		1			
15								2		2		2	
16										3			
17			1										
18								5		2		1	
19													

Note: Only the "dominate" (page 52) soils are used...A-4, A-6, A-7-6.

Table 3-17. (continued)

AASHTO DEPTH (ft)	A-1-A	A-1-B	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-4	A-5	A-6	A-7-5	A-7-6	A-8
18								5		2		1	
19												1	
20								3				1	
21													
22								3					
23													
24													
25													
26													
27													
28													
29											:		
30													
31											1		
32													
33													
34													
35													

The first layer ranges from 1 foot* to 12 feet, the second layer from 12 feet to 17 feet, and the third layer from 17 feet to 21 feet, as shown in Figure 3-3(b). All the upper boundaries of A-4, A-6, and A-7-6 soils extend to a depth of 12 feet. Further divisions of layers may be necessary. Referring to Figure 3-3(b) and Table 3-17, and for the sake of simplicity, the upper 12 foot layer is divided into half. Therefore a four layer soil profile system is established. The first layer is from 1 foot to 6 feet, the second layer from 6 feet to 12 feet, the third layer from 12 feet to 17 feet, and the fourth layer from 17 feet to 21 feet.

The soil characteristics presented in each layer are shrinkage limit in % (SL), natural moisture content (w_n), natural dry density (ρ_d), specific gravity (G), maximum dry density ($\rho_{d\ max}$), optimum moisture content (OMC), CBR soaked value at 100% maximum dry density (CBR SO 1), CBR soaked value at 95% maximum dry density (CBR SO2), and unconfined compressive strength (q_u). The sample distribution of each soil characteristic for a specific layer was then examined by using the procedures discussed in Section 3.3.1.a. All available data are used, regardless of their AASHTO classification units.

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*The top one foot of soil was regarded as top soil.

In addition to the soil characteristics mentioned above, estimates of sample distributions of SPT, preconsolidation pressure (p_c), cohesive strength intercept (c) and strength angle (ϕ) were examined and added to the statistical soil profiles (Table 3-18).

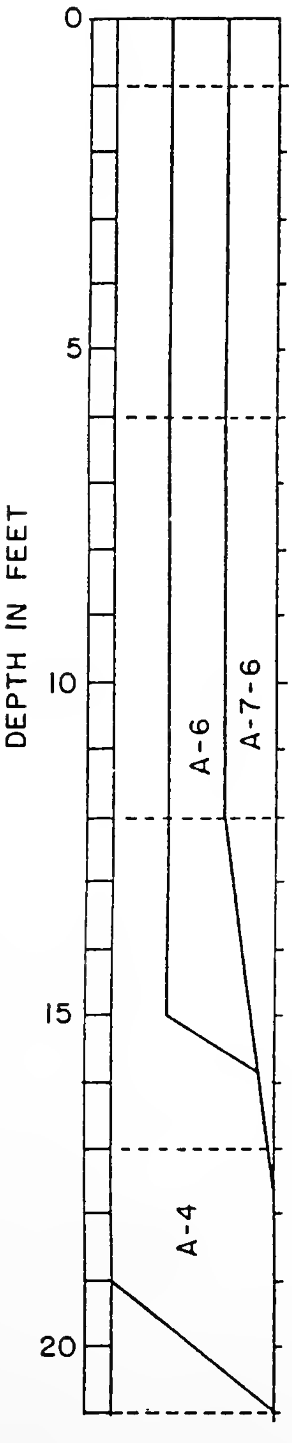
Thirty-eight statistical soil profiles were generated using the above procedures. The results are accumulated in Appendix A-111.

3.3.2.a. Regression Models

Regression analysis provides a conceptually simple method for investigating functional relationships among variables. In general, the first stage of the analysis is to select the variables to be included in the regression model. This is done based upon theory, on former examples, or by other procedures.

The most thorough approach, known as the all possible regression method, is to develop the regression of y (dependent variable) on every subset of the k x variables (independent variables). The major drawback of this method is the amount of computation. Another approach for selecting variables, and the one used in this study, is the stepwise regression method. For details refer to Chatterjee et al (18) and Draper et al (26). It is recommended (18) that the step-wise procedures be applied only to noncollinear data, and the order in which the variables enter or leave the equation not be interpreted

Table 3-18. Statistical Soil Profile for the Soil Association Wellston - Zanesville - Berks



Soil Association: Wellston-Zanesville-Berks

General Description: Sloping, well drained, silty Wellston and sloping, well drained, silty Zanesville with fragipans, both in wind-blown silts and weathered sandstone and shale, and steep Berks in weathered sandstone and shale.

Parent Material: Soils formed in residuum from siltstone, shale and sandstone bedrock.

Distribution:

Physiographic unit: Crawford Upland

Counties: Crawford, Dubois, Perry.

Topographic Characteristics:

Ground elevation (ft.):

mean:	541.23	s.d.:	110.85	median:	529.09
$t_{.25}$:	457.50	$t_{.75}$:	620.00	I.R.:	162.50
95% C.I. of median:			482.50~565.45	No. of cases:	59
minimum:	422.50		maximum:	803.80	

Ground water elevation (ft.):

mean:	532.87	s.d.:	107.88	median:	519.54
$t_{.25}$:	454.47	$t_{.75}$:	615.00	I.R.:	160.53
95% C.I. of median:			480.00~555.91	No. of cases:	59
minimum:	417.20		maximum:	784.80	

Water depth with relation to ground elevation (ft.):

mean:	8.36	s.d.:	7.12	median:	5.86
$t_{.25}$:	3.22	$t_{.75}$:	11.67	I.R.:	8.45
95% C.I. of median:			4.26~6.63	No. of cases:	59
minimum:	1.00		maximum:	37.00	

Table 3-18. (continued)

Engineering Characteristics:

Depth: 0.00' - 1.00'

Unified Classification: CL

PH: 4.5 - 5.0

Texture: silty clay loam (A-4, A-6, A-7-6)

Organic Material: not traceable

Depth: 0.00' - 6.00'

Unified Classification: CL, CH

PH: 4.5 - 5.0

Texture: clay (A-6, A-7-6), silty clay loam (A-4, A-6, A-7-6)

Organic Material: not traceable

Variable	Mean	S.D.	Median	$t_{.25}$	$t_{.75}$	I.R.	95% C. I. of Median	Minimum	Maximum	No. of Cases
SL (%)	18.55	3.81	18.13	15.24	21.18	5.94	17.11 -- 19.11	10.50	28.00	142
W_n (%)	22.17	4.77	21.87	18.33	25.00	6.67	20.55 -- 23.19	11.60	34.70	88
ρ_d (pcf)	103.73	7.12	103.42	100.13	108.25	8.12	101.58 -- 105.50	88.00	120.40	49
G	2.71	0.04	2.72	2.70	2.73	0.03	2.67 -- 2.74	2.60	2.75	10
ρ_{dmax} (pcf)	107.26	4.40	107.80	104.00	110.00	6.00	104.00 -- 110.00	100.80	116.00	14
OMC (%)	18.16	2.48	18.75	17.00	20.75	3.75	17.00 -- 20.75	14.30	22.00	14
CBR S01 (%)	7.13	3.39	7.00	5.00	10.00	5.00	5.00 -- 10.00	1.40	14.60	14
CBR S02 (%)	4.08	1.82	3.83	2.67	5.50	2.83	2.67 -- 5.50	1.30	7.60	14
q_u (Tsf)	0.86	0.67	0.67	0.35	1.10	0.75	0.40 -- 1.00	0.20	3.10	33

Table 3-18. (continued)

Depth: 6.00' - 12.00'

Unified Classification: CL, CH

PH: 4.5 - 5.0

Texture: clay (A-6, A-7-6), silty clay loam, sandy loam (A-4, A-6, A-7-6)
Organic Material:

Variable	Mean	S.D.	Median	$t_{.25}$	$t_{.75}$	I.R.	95% C. I. of Median	Minimum	Maximum	No. of Cases
SL (%)	16.50	4.35	16.32	12.71	19.15	6.44	15.00 -- 18.46	10.00	26.00	56
W_n (%)	23.36	9.14	21.00	16.67	30.00	13.33	17.78 — 26.67	12.00	43.30	27
ρ_d (pcf)	97.50	14.84	98.75	90.00	112.50	22.50	90.00 -- 112.50	71.00	117.60	14
G	2.69	0.06	2.70	2.67	2.74	0.07	— --	2.65	2.76	4
ρ_{dmax} (pcf)	104.67	4.20	105.00	102.50	107.50	5.00	— —	100.20	109.30	4
OMC (%)	20.05	2.84	20.00	18.00	22.50	4.50	— —	17.50	23.40	4
CBR SO1 (%)	4.07	1.12	4.55	3.50	4.65	1.15	-- —	2.40	4.70	4
CBR SO2 (%)	2.00	0.89	2.30	1.50	2.50	1.00	— —	0.70	2.70	4
q_u (Tsf)	1.15	0.86	0.90	0.57	1.60	1.03	0.57 — 1.60	0.50	3.20	8

Table 3-18. (continued)

Depth: 12.00' - 17.00'

Unified Classification: CL

PH: 4.5 - 5.00

Texture: clay (A-6, A-7-6), silty clay loam (A-4, A-6, A-7-6)

Organic Material: not traceable

Variable	Mean	S.D.	Median	$t_{.25}$	$t_{.75}$	I.R.	95% C. I. of Median	Minimum	Maximum	No. of Cases
SL (%)	18.87	4.90	18.90	15.67	22.36	6.69	16.67 -- 21.54	10.00	27.00	32
W_n (%)	27.65	5.74	28.50	23.75	32.50	8.75	23.75 -- 32.50	17.80	39.30	18
ρ_d (pcf)	94.29	7.50	96.25	90.83	101.25	10.42	90.00 -- 101.25	77.20	107.00	13
G	2.70	0.02	2.70	2.69	2.71	0.02	-- --	2.68	2.72	6
ρ_{dmax} (pcf)	107.00	--	107.00	--	--	--	-- --	--	--	1
OMC (%)	17.50	--	17.50	--	--	--	-- --	--	--	1
CBR S01 (%)	4.70	--	4.70	--	--	--	-- --	--	--	1
CBR S02 (%)	2.70	--	2.70	--	--	--	-- --	--	--	1
q_u (Tsf)	0.56	0.26	0.55	0.28	0.81	0.53	-- --	0.20	0.90	5

Table 3-18. (continued)

Depth: 17.00 - 21.00'

Unified Classification: CL, CL-ML

PH: 4.5 - 5.00

Texture: silty clay loam (A-4)

Organic Material: not traceable

Variable	Mean	S.D.	Median	$t_{.25}$	$t_{.75}$	I.R.	95% C. I. of Median	Minimum	Maximum	No. of Cases
SL (%)	21.65	2.61	22.00	19.50	23.12	3.62	21.00 -- 23.00	16.00	23.40	17
W_n (%)	27.04	3.26	27.73	25.68	29.77	4.09	25.91 -- 29.54	19.30	31.00	17
ρ_d (pcf)	97.96	4.82	97.50	94.00	101.67	7.67	94.00 -- 101.67	91.20	109.10	14
G	2.71	0.02	2.71	2.70	2.72	0.02	-- --	2.68	2.72	6
ρ_{dmax} (pcf)	--	--	--	--	--	--	-- --	--	--	0
OMC (%)	--	--	--	--	--	--	-- --	--	--	0
CBR SO1 (%)	--	--	--	--	--	--	-- --	--	--	0
CBR SO2 (%)	--	--	--	--	--	--	-- --	--	--	0
q_u (Tsf)	0.80	0.28	0.80	--	--	--	-- --	0.60	1.00	2

Table 3-18. (continued)

Strength Characteristics and Depth to Bedrock (Bedrk) Information:

County	Bedrk	Depth	SPT	p_c	c_v	Type	c	ϕ
13.	-0	2.00	3.	-0	-0	-0	-0	-0
13.	-0	2.00	-0	-0	-0	3.	.25	21.0
13.	-0	2.00	-0	-0	-0	3.	.15	24.0
13.	-0	3.00	7.	-0	-0	-0	-0	-0
13.	8.5	4.25	14.	-0	-0	-0	-0	-0
13.	-0	4.25	13.	-0	-0	-0	-0	-0
13.	-0	4.25	-0	-0	-0	3.	.50	31.0
13.	-0	4.75	28.	-0	-0	-0	-0	-0
13.	-0	5.50	5.	-0	-0	-0	-0	-0
13.	-0	6.00	6.	-0	-0	-0	-0	-0
13.	-0	6.75	5.	-0	-0	-0	-0	-0
13.	-0	7.00	7.	-0	-0	-0	-0	-0
13.	-0	7.00	4.	-0	-0	-0	-0	-0
13.	18.0	7.75	6.	-0	-0	-0	-0	-0
13.	-0	9.75	44.	-0	-0	-0	-0	-0
13.	-0	11.75	80.	-0	-0	-0	-0	-0
13.	-0	12.75	10.	-0	-0	-0	-0	-0
13.	-0	12.75	-0	-0	-0	3.	0	30.0
13.	24.0	13.25	25.	-0	-0	-0	-0	-0
13.	16.5	13.75	61.	-0	-0	-0	-0	-0

Table 3-18. (continued)

Strength Characteristics and Depth to Bedrock (Bedrk) Information:

County	Bedrk	Depth	SPT	p_c	c_v	Type	c	ϕ
13.	16.5	13.75	61.	-0	-0	-0	-0	-0
13.	-0	14.00	-0	-0	-0	3.	.10	20.0
13.	22.5	14.75	-0	-0	-0	-0	-0	-0
13.	16.5	15.25	31.	-0	-0	-0	-0	-0
13.	-0	15.50	7.	-0	-0	-0	-0	-0
13.	-0	16.00	-0	-0	-0	3.	.25	21.0
13.	25.0	16.75	7.	-0	-0	-0	-0	-0
13.	-0	18.25	45.	-0	-0	-0	-0	-0
19.	-0	3.50	-0	1.60	-0	-0	-0	-0
19.	-0	6.00	-0	1.65	-0	-0	-0	-0
19.	-0	6.00	-0	3.20	-0	-0	-0	-0
19.	-0	6.00	-0	-0	-0	-0	0	32.6
19.	-0	6.00	-0	4.40	-0	-0	-0	-0
19.	-0	6.00	-0	-0	-0	3.	0	32.6
19.	-0	6.00	-0	-0	-0	3.	0	32.6
19.	9.5	7.25	31.	-0	-0	-0	-0	-0
19.	-0	7.50	5.	-0	-0	-0	-0	-0
19.	-0	7.75	3.	-0	-0	-0	-0	-0

Table 3-18. (continued)

Strength Characteristics and Depth to Bedrock (Bedrk) Information:

County	Bedrk	Depth	SPT	p_c	c_v	Type	c	ϕ
62.	-0	.20	-0	3.00	-0	-0	-0	-0
62.	9.0	1.00	3.	-0	-0	-0	-0	-0
62.	5.5	1.75	18.	-0	-0	-0	-0	-0
62.	-0	1.75	64.	-0	-0	-0	-0	-0
62.	-0	3.00	-0	2.15	-0	-0	-0	-0
62.	-0	3.25	-0	2.10	-0	-0	-0	-0
62.	-0	4.00	-0	1.50	-0	-0	-0	-0
62.	5.5	4.25	28.	-0	-0	-0	-0	-0
62.	9.0	4.25	16.	-0	-0	-0	-0	-0
62.	-0	4.75	2.	-0	-0	-0	-0	-0
62.	9.3	5.75	18.	-0	-0	-0	-0	-0
62.	6.5	5.75	21.	-0	-0	-0	-0	-0
62.	-0	5.75	4.	-0	-0	-0	-0	-0
62.	-0	5.75	2.	-0	-0	-0	-0	-0
62.	8.5	6.75	25.	-0	-0	-0	-0	-0
62.	-0	7.50	-0	2.40	-0	-0	-0	-0
62.	-0	7.75	-0	6.30	-0	-0	-0	-0
62.	-0	8.00	5.	-0	-0	-0	-0	-0

Table 3-18. (continued)

Strength Characteristics and Depth to Bedrock (Bedrk) Information:

County	Bedrk	Depth	SPT	P_c	c_v	Type	c	ϕ
19.	-0	10.25	25.	-0	-0	-0	-0	-0
19.	-0	11.00	-0	2.15	3.60	-0	-0	-0
19.	-0	12.75	9.	-0	-0	-0	-0	-0
19.	-0	12.75	7.	-0	-0	-0	-0	-0
19.	-0	16.00	-0	2.00	-0	-0	-0	-0
19.	32.8	17.00	-0	1.35	3.60	-0	-0	-0
19.	-0	18.00	-0	-0	-0	-0	0	29.3
19.	-0	18.00	-0	-0	-0	-0	0	29.3
19.	-0	18.00	-0	-0	-0	-0	0	29.3
19.	-0	19.25	11.	-0	-0	-0	-0	-0
19.	29.5	19.25	-0	-0	-0	2.	0	36.0
19.	29.5	19.25	-0	-0	-0	2.	0	36.0
19.	29.5	19.25	-0	-0	-0	2.	0	36.0
19.	-0	19.50	-0	3.10	-0	-0	-0	-0
19.	29.5	23.80	-0	-0	-0	2.	0	33.0
19.	29.5	23.80	-0	-0	-0	2.	0	33.0
19.	-0	23.80	-0	-0	-0	2.	0	33.0

Table 3-18. (continued)

Strength Characteristics and Depth to Bedrock (Bedrk) Information:

County	Bedrk	Depth	SPT	p_c	c_v	Type	c	ϕ
62.	-0	8.25	24.	-0	-0	-0	-0	-0
62.	-0	10.75	73.	-0	-0	-0	-0	-0
62.	-0	14.25	51.	-0	-0	-0	-0	-0
62.	22.3	14.75	38.	-0	-0	-0	-0	-0
62.	-0	18.25	9.	-0	-0	-0	-0	-0
62.	18.0	19.25	50.	-0	-0	-0	-0	-0

Note: 1. (-0) unknown

2. County and Type as coded.

3. Bedrk in ft. Depth in ft.

p_c in Tsf. c_v in ft^2/mth.

c in Tsf. ϕ in degree.

as reflecting the relative importance of the variables.

A question arises while entering the variables to formulate a regression model, as to the form of each variable, i.e., should it enter the model as an original variable x, or as some transformed variable such as x^2, log x, or a combination of both. If from an examination of scatter plots of y against x the relationship between y and x appears to be nonlinear, appropriate transformations of the data are introduced to produce linearity (18). In this study, all variables, their possible transformations, and their combinations were included in the step-wise procedure for selecting variables, so long as they were not collinear.

The variables were selected to minimize the MSE (mean square due to error) of the prediction. As a large value of R^2 or a significant t statistic does not insure that the data were well fitted (3), a careful residual analysis was also made. The procedure to reduce the number of independent variables was to compare the full model (FM) and reduced model (RM) by using the F-statistic. For details of this procedure refer to (18).

It was believed that the soil was more homogeneous in a small geologic or pedologic unit. Therefore, the regression models were established on these units. It was often found that for a given dependent variable the regression models generated in this way used different sets of independent variables for various locations. It

seems unwise to conclude that these differences are caused by soil differences alone.

The effects of soil location and genesis, viz., physiographic region and parent material, were investigated by employing the statistical technique of using qualitative variables as regressors (18). In order to do so, the qualitative variables were represented by dummy variables which take on only two values, usually zero and one. These two values designated whether the observation belonged in one of two possible categories. Accordingly, the number of these variables required was one less than the number of categories in a grouping unit. Reference (53) shows that for Indiana the physiographic regions are coded from 1 to 12 and parent materials from 1 to 13. The dummy variables indicators were set up as follows:

$$x_i \begin{cases} = 1 & \text{if the soil sample is taken from the physiographic region coded as } i \\ = 0 & \text{otherwise} \end{cases}$$

where $i = 1, 2, 3, \ldots, 11$;

and

$$z_j \begin{cases} = 1 & \text{if the soil sample is derived from the parent material coded as } j \\ = 0 & \text{otherwise} \end{cases}$$

where $j = 1, 2, 3, \ldots, 12$.

For the soil sample taken from the physiographic region coded as 13, let $x_1 = x_2 = x_3 = \ldots = x_{12} = 0$. And for the soil sample derived from the parent material coded as

12, let $z_1 = z_2 = z_3 = \ldots = z_{11} = 0$.

Assume that the following relationship exists:

$$C_c = a_0 + a_1 w_n + a_2 e_o + a_3 w_L$$

where C_c is the compression index, w_n the natural moisture content (%), e_o the initial void ratio, and w_L the liquid limit (%). To investigate how these two soil grouping units affect this relationship singly or in combination, the F-statistics were employed for making comparisons of the following models:

Model 1: $C_c = a_0 + a_1 w_n + a_2 e_o + a_3 w_L$

Model 2: $C_c = a_0' + a_1' w_n + a_2' e_o + a_3' w_L + a_4' x_1 + a_5' x_2 + \ldots + a_{15}' x_{12}$

Model 3: $C_c = a_0'' + a_1'' w_n + a_2'' e_o + a_3'' w_L + a_4'' z_1 + a_5'' z_2 + \ldots + a_{14}'' z_{11}$

and

Model 4: $C_c = a_o''' + a_1''' w_n + a_2''' e_o + a_3''' w_L + a_4''' x_1 + a_5''' x_2 + \ldots + a_{15}''' x_{12} + a_{16}''' z_1 + a_{17}''' z_2 + \ldots + a_{26}''' z_{11}$.

3.3.2.b. The Application of Regression Models

The regression models were used to correlate soil design parameters, such as those of compaction, consolidation, and strength, with soil index properties. The following example illustrates how to apply the procedures described in the previous section to arrive at the regression model for maximum dry density ($\rho_{d\ max}$) using index properties.

Table 3-19 shows the correlation coefficients (R) between the $\rho_{d\ max}$ and selected index properties. It shows that the $\rho_{d\ max}$ is well correlated with w_L and w_p. Scatter plots are shown in Figures 3-4 and 3-5. In the presentation of the data, the solid line represents the best fit line while the dashed lines define the boundaries of 95% population confidence interval. Examinations of these two scatter plots indicate that seemingly curvilinear relationships exist between $\rho_{d\ max}$ and w_L and w_p. There should also be an examination of whether the interaction between w_L and w_p, i.e., $w_L \cdot w_p$, has any significant

Table 3-19 Correlation Table for Maximum Dry Density ($\rho_{d\ max}$) Versus Some Index Properties

Coefficient of Correlation (R)	Natural Moisture Content, %, (w_n)	Natural Dry Density, pcf, (ρ_d)	% Sand by Weight (sand)
$\rho_{d\ max}$(pcf)	- 0.525 (n = 241)	0.714 (n = 20)	0.201 (n = 642)

Table 3-19 (continued)

	% Silt by Weight, (silt)	Plastic Limit, % (w_p)	Liquid Limit %, (w_L)
	-0.243 (n = 668)	-0.665 (n = 623)	-0.720 (n = 625)

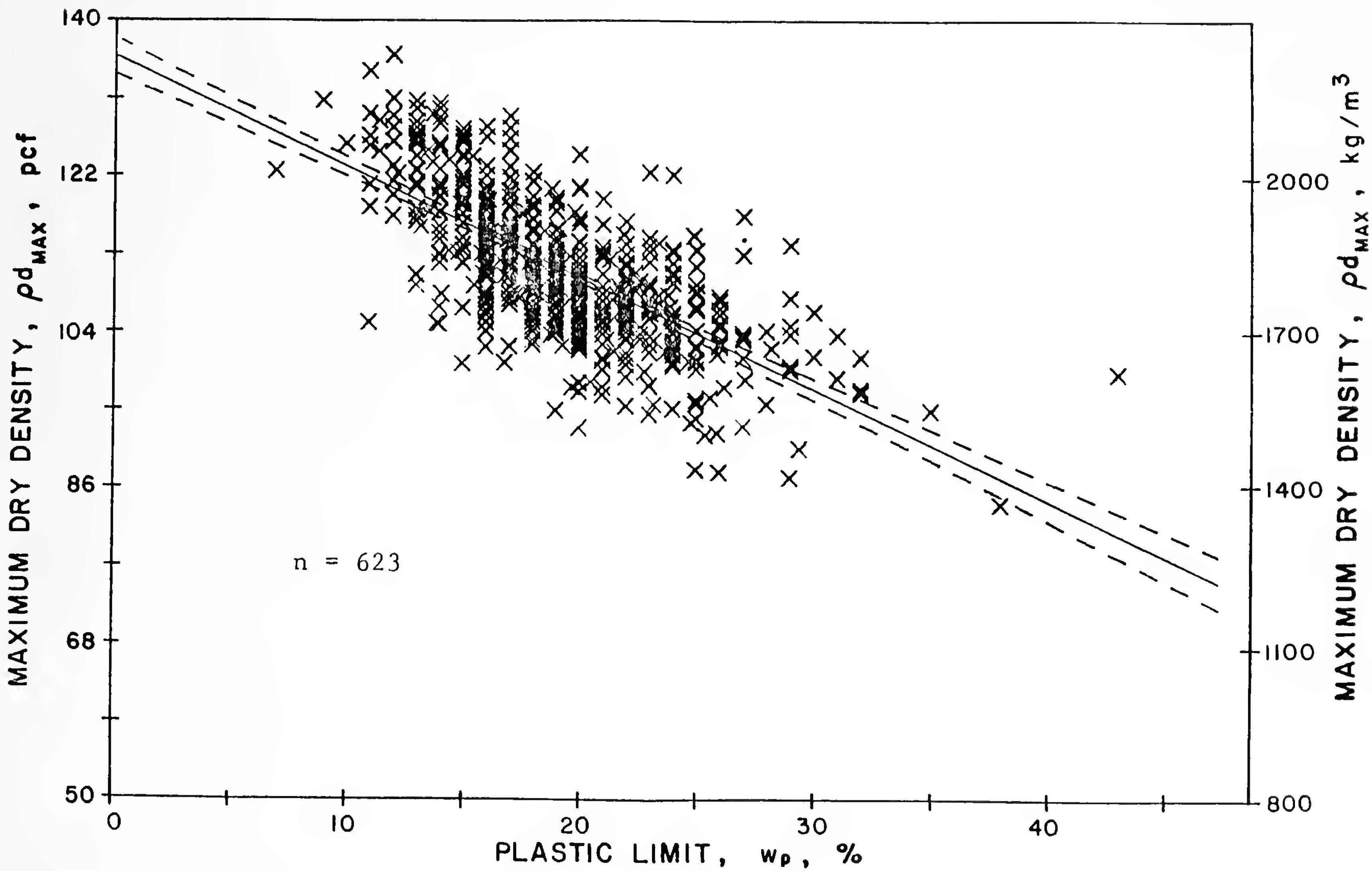


Figure 3-4. Maximum Dry Density vs Plastic Limit, Indiana Soils

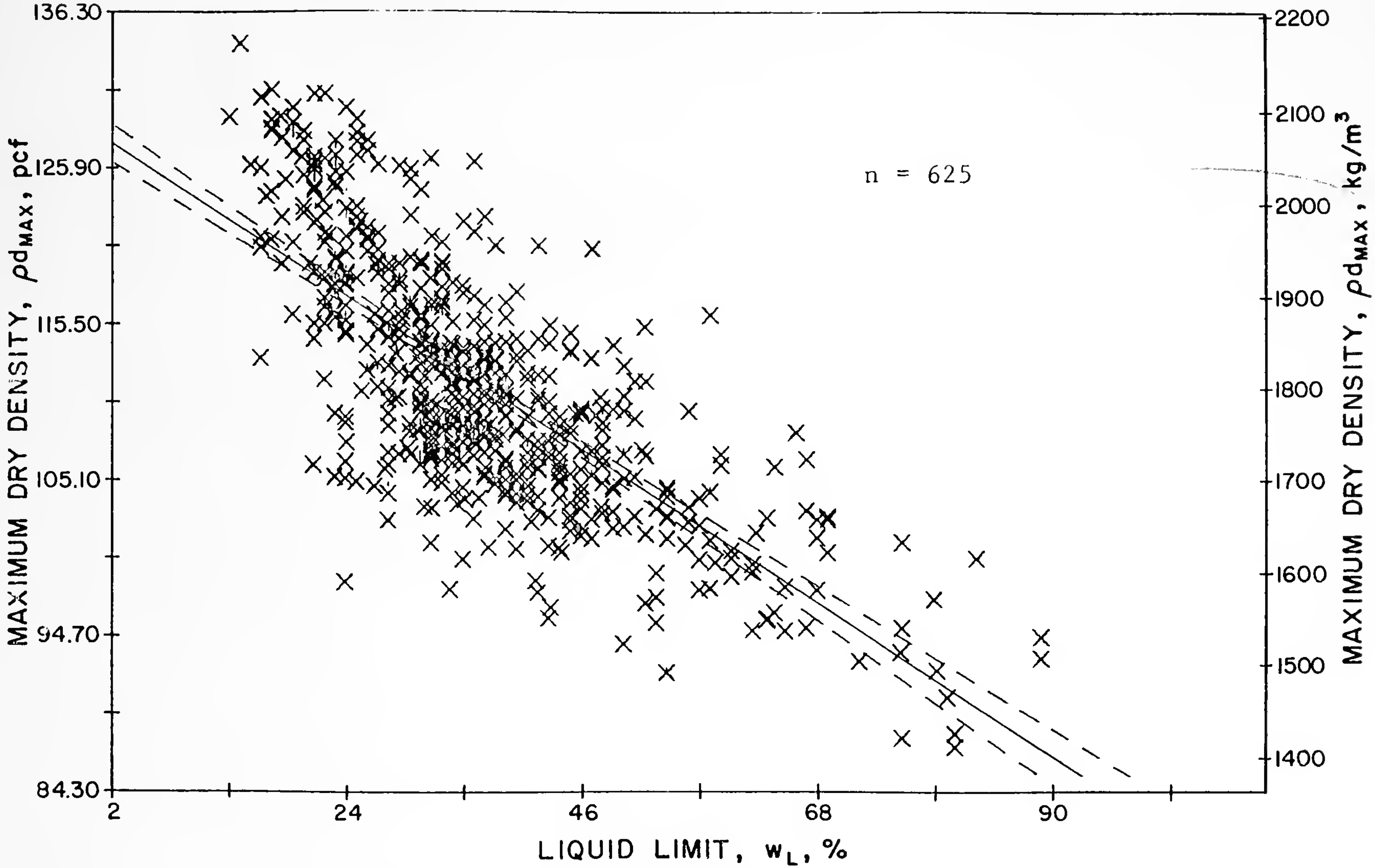


Figure 3-5. Maximum Dry Density vs. Liquid Limit, Indiana Soils

effect on $\rho_{d\ max}$. Therefore, the following independent variables enter the step-wise regression program for consideration: w_L, w_L^2, w_p, w_p^2, $w_L \cdot w_p$, % sand and % silt. Since there were not enough corresponding data available for w_n and ρ_d, they do not enter the program as regressor candidates. The results follow.

Model 1: $\hat{\rho}_{d\ max} = - 0.463\ w_L + 128.337$, for which $|R|$ = 0.743, standard deviation of estimate = 5.651, and n = 601.

ANOV (analysis of variance) table

source	d.f.	s.s.	m.s.
regression	1	23619.122	23619.122
errors	599	19131.922	31.940

Model 2: $\hat{\rho}_{d\ max} = - 0.465\ w_L - 0.138$ silt $+ 133.995$, for which $|R|$ = 0.791, standard deviation of estimate = 5.176, and n = 601.

ANOV table

source	d.f.	s.s.	m.s.
regression	2	26726.969	13363.485
errors	598	16024.075	26.796

Model 3: $\hat{\rho}_{d\ max} = - 0.371\ w_L - 0.105$ silt $- 0.396\ w_p + 136.794$, for which $|R|$ = 0.803, standard deviation of estimate = 5.047, and n = 601.

ANOV table

source	d.f.	s.s.	m.s.
regression	3	27544.092	9181.364
errors	397	15206.952	25.472

Model 4: $\hat{\rho}_{d\ max} = -\ 0.554\ w_L - 0.0900\ \text{silt} - 0.277\ w_p + 0.00849\ w_L \cdot w_p + 142.888$, for which $|R| = 0.808$, standard deviation of estimate $= 4.984$, and $n = 601$.

ANOV table

source	d.f.	s.s.	m.s.
regression	4	27884.774	6971.193
errors	596	14866.270	24.943

Model 5: $\hat{\rho}_{d\ max} = -\ 0.587\ w_L - 0.0898\ \text{silt} - 0.608\ w_p + 0.0100\ w_L w_p - 0.00430\ w_p^2 + 142.344$, for which $|R| = 0.808$, standard deviation of estimate $= 4.998$, and $n = 601$.

ANOV table

source	d.f.	s.s.	m.s.
regression	5	27890.335	5578.067
errors	595	14860.709	24.976

Model 6: $\hat{\rho}_{d\ max} = -\ 0.591\ w_L - 0.0897\ \text{silt} - 0.606\ w_p + 0.0122\ w_L w_p - 0.00648\ w_p^2 - 0.000470\ w_L^2 + 142.404$,

for which $|R| = 0.808$, standard deviation of estimate = 5.001, and n = 601.

ANOV table

source	d.f.	s.s.	m.s.
regression	6	27892.792	4648.799
errors	594	14858.252	25.014

Model 7: $\hat{\rho}_{d\ max} = -\ 0.588\ w_L - 0.0858\ \text{silt} - 0.608\ w_p + 0.0126\ w_L w_p - 0.00683\ w_p^2 - 0.000546\ w_L^2 + 0.00585\ \text{sand} + 142.000$, for which $|R| = 0.808$, standard deviation of estimate = 5.005, and n = 601.

ANOV table

source	d.f.	s.s.	m.s.
regression	7	27894.718	3984.960
errors	593	14856.326	25.053

The F-statistic tests were employed to reduce the number of independent variables as follows:

Case i, Model 7 vs 4

H_o: coefficients of sand, $w_L{}^2$, $w_p{}^2 = 0$, H_A: not H_o.

$$F_{3,583} = \frac{(27894.718 - 27884.774)/(7-4)}{25.053}$$

$$= 0.132 < 8.54 \text{ at } 5\% \text{ level.}$$

H_o is accepted, i.e., the variables of sand, $w_L{}^2$, and $w_p{}^2$ do not add significant information to Model 4.

case ii, Model 4 vs 3:

H_o: coefficient of $w_L w_p = 0$, H_A: not H_o

$$F_{1,596} = \frac{(27884.774 - 27544.092)/(4-3)}{24.943}$$

$$= 13.658 > 3.86 \text{ at 5\% level.}$$

H_o is rejected, i.e., the variable $w_L w_p$ does add significant information to Model 3.

case iii, Model 3 vs 2:

H_o: coefficient of $w_p = 0$, H_A: not H_o

$$F_{1,597} = \frac{(27544.092 - 26726.969)/(3-2)}{25.472}$$

$$= 32.079 > 3.86 \text{ at 5\% level.}$$

H_o is rejected.

Case iv, Model 2 vs 1:

H_o: coefficient of silt = 0, H_A: not H_o

$$F_{1,598} = \frac{(26726.969 - 23619.122)/(2-1)}{26.796}$$

$$= 115.982 > 3.86 \text{ at 5\% level}$$

H_o is rejected.

Hence, the final model was in the form of:

$$\hat{\rho}_{d\ max} = -\ 0.554\ w_L - 0.0900\ \text{silt} - 0.727\ w_p + 0.00849\ w_L w_p + 142.888, \text{ Equation 3.3,}$$

for which $|R| = 0.808$, standard deviation of estimate $= 4.994$, and $n = 601$. For a given predicted $\rho_{d\ max}$ ($\hat{\rho}_{d\ max}$) about 68% of sample observations (measured $\rho_{d\ max}$) fall in the range of $\hat{\rho}_{d\ max} + 4.994$ and $\hat{\rho}_{d\ max} - 4.994$.

To investigate the effects of physiographic regions and parent materials on Equation 3.3 the dummy variable indicators were set up as follows:

$$x_i \begin{cases} = 1 \text{ if the soil sample is taken from the physiographic region coded as } i \\ = 0 \text{ otherwise} \end{cases}$$

where i = 1, 2, 3, ..., 11*

and

$$z_j \begin{cases} = 1 \text{ if the soil sample is derived from the parent material coded as } j \\ = 0 \text{ otherwise} \end{cases}$$

where j = 1, 2, 3, ..., 11.

Adding these dummy variable indicators to the regression model (Equation 3.3) for further analysis, the regression models were developed as follows.

Model A: $\hat{\rho}_{d\ max} = -\ 0.554\ w_L - 0.0900\ \text{silt} - 0.727\ w_p + 0.00849\ w_L w_p + 142.888$, for which $|R| = 0.808$, standard deviation of estimate = 4,994, and n = 601.

ANOV table

source	d.f.	s.s	m.s.
regression	4	27884.774	6971.193
errors	596	14866.270	24.943

*There were no soil samples available from the Maumee Lacustrine Section, coded as 12.

Model B: $\hat{\rho}_{d\ max} = -\ 0.546\ w_L - 0.0903\ \text{silt} - 0.680\ w_p$
$+ 0.00818\ w_L w_p + 0.540\ x_1 + 0.292\ x_2 + 4.450\ x_3$
$- 1.992\ x_4 + 0.488\ x_5 - 1.374\ x_6 + 0.368\ x_7 - 0.138\ x_8$
$- 1.574\ x_9 - 0.136\ x_{10} + 0.755\ x_{11} + 141.852$

for which $|R| = 0.814$, standard deviation of estimate = 4.960, and n = 601.

ANOV table

source	d.f.	s.s.	m.s.
regression	15	28359.223	1890.615
errors	585	14391.821	24.601

Model C: $\hat{\rho}_{d\ max} = -\ 0.564\ w_L - 0.0920\ \text{silt} - 0.695\ w_p$
$+ 0.00873\ w_L w_p - 2.625\ z_1 - 3.912\ z_2 - 6.532\ z_3$
$- 4.281\ z_4 - 2.706\ z_5 - 4.147\ z_6 - 2.717\ z_7$
$- 3.0270\ z_8 - 4.314\ z_9 - 3.390\ z_{10} - 4.449\ z_{11}$
$+ 146.137$

for which $|R| = 0.813$, standard deviation of estimate = 4.976, and n = 601.

ANOV table

source	d.f.	s.s.	m.s.
regression	15	28263.429	1884.228
errors	585	14487.615	24.765

Model D: $\hat{\rho}_{d\ max} = -\ 0.563\ w_L - 0.0938\ \text{silt} - 0.654\ w_p$
$+ 0.00828\ w_L w_p + 0.240\ x_1 + 0.191\ x_2 + 4.516\ x_3$
$- 2.677\ x_4 - 0.478\ x_5 - 2.453\ x_6 - 0.664\ x_7 - 0.447\ x_8$

$$- 2.501\ x_9 + 0.569\ x_{10} + 0.281\ x_{11} - 2.480\ z_1 - 4.761\ z_2 - 7.246\ z_3 - 3.746\ z_4 - 3.715\ z_5 - 5.321\ z_6 - 3.847\ z_7 - 4.190\ z_8 - 4.684\ z_9 - 3.329\ z_{10} - 3.254\ z_{11}$$

for which $|R| = 0.819$, standard deviation of estimate = 4.946, and n = 601.

ANOV table

source	d.f.	s.s.	m.s.
regression	26	28710.798	1104.261
errors	574	14040.246	24.460

The F statistic tests were again used to examine the effects of physiographic regions and parent materials on the Model A, i.e., Equation 3.3

Case i: Model D vs A

H_o: coefficient of x's and z's = 0, H_A: not H_o.

$$F_{22,574} = \frac{(28710.798 - 27884.774)/(26-4)}{24.460}$$

$$= 1.535 < 1.560 \text{ at 5\% level.}$$

H_o is accepted, i.e., neither physiographic regions nor parent materials add any significant information on Model A.

Case ii: Model B vs A

H_o: coefficient of x's = 0, H_A: not H_o.

$$F_{11,585} = \frac{(28359.223 - 27884.774)/(15-4)}{24.601}$$

$$= 1.753 < 1.810 \text{ at 5\% level.}$$

H_o is accepted, i.e., physiographic regions do not add significant information on Model A.

Case iii: Model C vs A

H_o: coefficient of z's = 0, H_A: not H_o:

$$F_{11,585} = \frac{(28263.429 - 27884.774)/(15-4)}{24.765}$$

$$= 1.390 < 1.810 \text{ at } 5\% \text{ level}$$

H_o is accepted, i.e., parent materials do not add significant information on Model A.

Therefore, equation 3.3 was the final model for maximum dry density ($\rho_{d\ max}$). For the soil variable of compression index (C_c) it was found that effects of both physiographic regions and parent materials do add significant information statistically to the regression model of C_c on index properties. The regression model was found to be

$$\begin{aligned}\hat{C}_c = & - 0.151 + 0.00326\ w_n + 0.191\ e_o + 0.00325\ w_L \\ & + 0.0162\ x_1 - 0.0110\ x_2 + 0.0208\ x_3 + 0.0296\ x_4 \\ & + 0.0120\ x_5 - 0.0110\ x_6 + 0.0365\ x_7 + 0.0351\ x_8 \\ & + 0.0646\ x_9 + 0.0649\ x_{10} - 0.0.594\ x_{11} - 0.0245\ z_1 \\ & - 0.0313\ z_2 - 0.00987\ z_3 - 0.0917\ z_4 - 0.121\ z_6 \\ & - 0.0292\ z_7 - 0.0667\ z_8 + 0.00841\ z_9 - 0.0418\ z_{10} \\ & - 0.00884\ z_{11}\end{aligned}$$

Equation 3.4*

for which $|R| = 0.952$, standard deviation of estimate $= 0.0670$, and $n = 302$.

*Note that there were no soil sample taken from the Maumee Lacustrine section, coded as 12 in the group of physiographic regions, nor from loamy Wisconsin age glacial till, coded as 5 in the group of parent materials.

The Crawford Upland soils are coded as 7, i.e., x_7, in the group of physiographic regions, and those of soil association Wellston-Zanesville-Berks which is the residuum from siltstone, shale and sandstone are coded as 10, i.e., z_{10}, in the group of parent materials. With $x_7 = 1$, $z_{10} = 1$, and the rest of dummy variables indicators as zero, Equation 3.4 becomes

$$\hat{C}_c = -\ 0.151 + 0.00326\ w_n + 0.191\ e_o + 0.00325\ w_L + 0.0365 - 0.0418$$

or

$$= -\ 0.156 + 0.00326\ w_n + 0.191\ e_o + 0.00325\ w_L.$$

Figures 3-6 and 3-7 show the scatter plots with regression lines and 95% population confidence intervals for the measured $\rho_{d\ max}$ ($\rho_{d\ max}$) and predicted $\rho_{d\ max}$ ($\hat{\rho}_{d\ max}$) using Equation 3.3, and the measured C_c (C_c) and predicted C_c ($\hat{C}_c$) using Equation 3.4. In the presentation of the data, the solid line represents the best fit line, while the dashed lines define the boundaries of 95% population confidence intervals. All the regression models generated in this work are collected in Appendix A-IV.

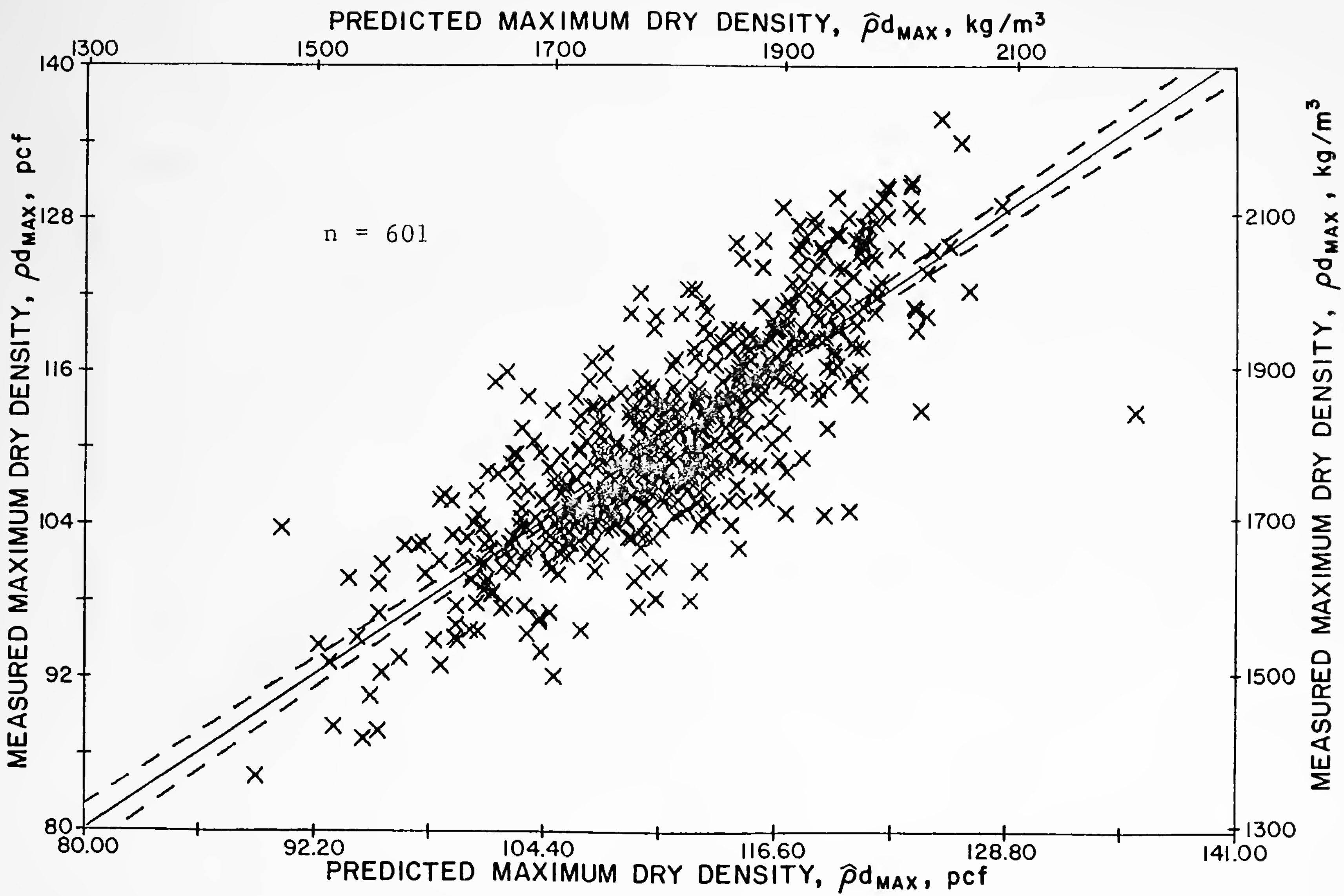


Figure 3-5. Measured Maximum Dry Density vs. Predicted Maximum Dry Density, Indiana Soils

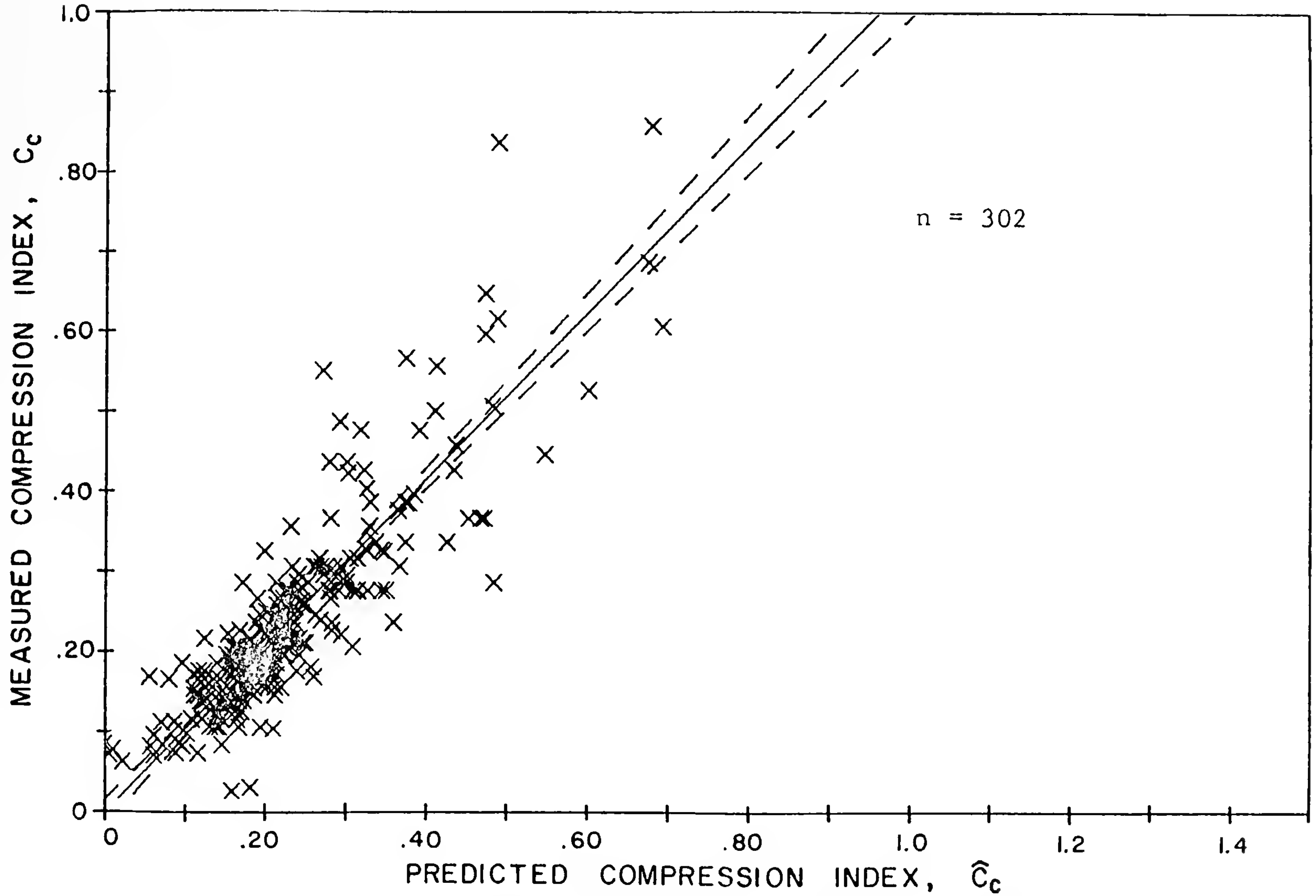


Figure 3-7. Measured Compression Index vs Predicted Compression Index, Indiana Soils

CHAPTER 4 DISCUSSIONS OF RESULTS

4.1. General

In this study both median and regression models were developed for statistical forecasting. Predictions are extrapolations into the future of features shown by relevant data in the past. Therefore, a considerable population of values for the dependent variable is required. Another basic requirement for prediction is the existence of a stable data structure. The trend of the data and the statistical variation about the trend must be stable, which can be detected by the confidence intervals and interquartile range for a median model, or the standard deviation of estimate and multiple correlation coefficient ($|R|$) for a regression model. A large difference between the confidence intervals, or a large value of interquartile range for a median model, or a large standard deviation of estimate for a regression model, indicates that the data structure is not stable. Either more data or a change in grouping unit is needed for better prediction.

4.2. The Median Model and Soil Variability

In this study soils were grouped by physiographic regions, AASHTO classifications, soil associations, or a

combination of these. The sample distributions of soil characteristics were studied according to these groups. The median of the sample distribution of a soil characteristic was used as the design value.

Technically, the grouping layout used in this study is known as either a one-way classification or a two-way classification. The mathematical model for a median in a one-way classification (Table 4-1) is expressed as

$$\hat{\theta}_{ij} = \theta + \tau_j + e_{ij},\ i = 1, \ldots, n_j,\ j = 1, \ldots, k,$$

Table 4-1 One-way Classification of Soil Variable ($\hat{\theta}_{ij}$)

Soil Groups					
1	2	.	.	.	k
$\hat{\theta}_{11}$	$\hat{\theta}_{12}$				$\hat{\theta}_{1k}$
$\hat{\theta}_{21}$	$\hat{\theta}_{22}$				$\hat{\theta}_{2k}$
.	.				.
.	.				.
.	.				.
$\hat{\theta}_{n_1 1}$	$\hat{\theta}_{n_2 2}$				$\hat{\theta}_{n_k k}$

Data: The data consist of $N = \sum_{j=1}^{k} n_j$ samples with n_j samples from the j-th soil group, $j = 1, \ldots, k$.

where $\hat{\theta}_{ij}$ is the overall median, τ_j the deviation due to the effect of soil group j, and e_{ij} the random error. If $\tau_1 = \tau_2 = \ldots = \tau_k = 0$, the deviations between the medians of the k-group soil samples are due only to the random

errors. However, if $\tau_j \neq \tau_{j+1}$, then $\hat{\theta}_{ij} \neq \hat{\theta}_{i\ j+1}$, viz., the deviation between the medians of the j-th and (j+1)-th soil sample groups is due to the groupings. The test hypotheses can be set up as follows:

H_o: The medians of the k populations are equal.

H_A: At least one of the populations has a median different from the others.

In the following, the nonparametric sample comparison methods, such as the median test (101), the Kolmogorov-Smirnov two sample test (42, 101), the Mann-Whitney U-Wilcoxon rank sum W test (42, 101) the Wald-Wolfowitz run test (42, 101), the extended median test (101), and the Kruskal-Wallis one-way analysis of variance (ANOV) by ranks test (42, 101), are employed to investigate the group effects on the soil characteristics. For the details of these tests refer to Bradley (14), Hollander et al (42), and Siegel (101).

4.2.a. Topographic Characteristics Versus Physiographic Regions

Topographic characteristics were grouped according to physiographic regions. The test results, as shown in Table C-1 to C-14, Appendix C, verify that the topographic features, such as the ground elevations and ground-water elevations, vary with physiographic regions. These test results, together with the numerical information of topographic characteristics as shown in Table A-I-1 and A-I-3,

Appendix A, can also be used to compare the overall topographic features between two physiographic regions. For example, Table C-2 implies that the median of ground elevations in Tipton Till Plain is greater than that of Dearborn Upland. This is further confirmed by the numerical results as shown in Table A-I-1, Appendix A, in which the medians of the ground elevations of the Tipton Till Plain and Dearborn Upland are 776.64 ft. and 737.67 ft. respectively. Table A-I-1 also shows that the interquartile range (IR) of the Tipton Till Plain is 129.03 ft. and that of the Dearborn Upland is 199.26 ft. This indicates that the general topography of the Dearborn Upland is more rugged than that of the Tipton Till Plain.

Table C-6 and Table A-I-1 also indicate that the general topography of the Crawford Upland is relatively more elevated and rugged than that of the Wabash Lowland.

It is emphasized that the observations of topographic characteristics are not uniformly distributed throughout a physiographic region. The soil data distribution map (Figure 3-1) should be consulted before attempting any such interpretations.

4.2.b. Remolded Soil Characteristics Versus Physiographic Regions and AASHTO Classifications

As mentioned in Section 3.3.1.b. the sample distributions of remolded soil characteristics were studied according to AASHTO classifications for each physiographic

region. Technically, this is a two-way classification layout, i.e., the grouping effects are due to a combination of AASHTO classifications and physiographic regions. These may be investigated by first examining the effects of physiographic regions, followed by an examination of the effects of AASHTO classifications. The test results, as shown in Tables C-15 to C-24, indicate that the remolded soil characteristics vary with either grouping unit, as exemplified by maximum dry density ($\rho_{d\ max}$) and optimum moisture content (OMC). Therefore, it is reasonable to use these grouping units.

The results in Tables C-16* and C-21* imply that: the median of $\rho_{d\ max}$ of A-4 soils > the median of $\rho_{d\ max}$ of A-6 soils > the median of $\rho_{d\ max}$ of A-7-6 soils; and the median of OMC of A-7-6 soils > the median of OMC of A-6 soils > the median of OMC of A-4 soils. The more plastic soils have lower $\rho_{d\ max}$ and higher OMC values. These facts are further verified by use of the two-sample comparison tests, as shown in Tables C-17 to C-19 and Tables C-20 to C-24, and the numerical results in Tables A-II-1 to A-II-36.

4.2.c. Soil Characteristics Versus Soil Associations

The layout for the generation of a statistical soil profile involves a breakdown of the data for each soil characteristic by physiographic regions, and then by soil

* The reader will need to use the code on page 460 to interpret these tables.

associations. Finally they are grouped into proper layer systems. The test results, as illustrated in Tables C-25 and C-26, show that there are differences of soil characteristics for the groups of soil associations, as exemplified by natural dry density (ρ_d) and unconfined compressive strength (q_u). However, due to the wide scatter of sample frequencies among cells, i.e., soil associations units, further verification is required.

Tables C-29 and C-30 indicate that the natural dry density (ρ_d) does not vary significantly either within the group of soil associations coded as 3, 4, 7 and 9 or within the group of soil associations coded as 105, 106, and 107. This may mean either that: (1) the grouping unit is too refined, or (2) the grouping unit is not refined enough. In the latter case a subgrouping unit, soil series, should be perhaps consulted. This subject needs further research.

4.2.d. Soil Series as a Grouping Unit

Tables C-31 and C-32 show the multiple comparison tests of soil characteristics versus soil series, as exemplified by ρ_d and q_u. The wide scatter and the shortage of data can be seen from these two tables. In spite of the small sample sizes of soil characteristics, the comparison tests were made. Tables C-33 through C-35 indicate that the soil characteristics do not vary significantly with soil series. Therefore, the soil series as a grouping unit is indicated to be no better than the soil association. Further study would be required to establish this as a firm conclusion.

4.3. Regression Models and Correlations

Regression models were used to correlate the soil design parameters, such as those of compaction, consolidation, and strength, with index properties. In Appendix A each regression equation is represented, together with its correlation coefficient (R) or multiple correlation coefficient (|R|), standard deviation (or error) of estimate (s.d. of est.), and the number of cases (n). The (s.d. of est.) is important, as it represents the variation of estimate ($\hat{y}$), viz. 68% of sample observations (y) fall in the range of $\hat{y}$ - (s.d. of est.) and $\hat{y}$ + (s.d. of est.). Specific results are presented below.

4.3.a. Compaction Parameters

The correlations of maximum dry density ($\rho_{d\ max}$) and optimum moisture content (OMC) versus plasticity characteristics, as shown in Appendix A, indicate that as the liquid limit or plastic limit increases, the OMC increases but $\rho_{d\ max}$ decreases. Also, as the OMC increases, the $\rho_{d\ max}$ decreases. The explanation of these correlations has been previously discussed in Section 2.2.2.a .

The CBR value is regarded as an indirect measure of strength of compacted soil. The strength of a standard compacted soil is a function of its (maximum) dry density and (optimum) moisture content as proposed by Jorgenson (47) and Weitzel (120). Accordingly, the CBR value is a

function of plastic characteristics which is evidenced by the regression equation for CBR at 100% maximum dry density as shown in Appendix A-IV. A relationship between CBR values at 100% and 95% maximum dry densities is also developed and presented in Appendix A.

4.3.b. Consolidation Parameters

It is found (Appendix A-IV) that the compression index (C_c) increases with either liquid limit (w_L), or natural moisture content (w_n), or initial void ratio (e_o). The mathematical relationship between compression ratio (C_r') and compression index (C_c) is $C_r' = \frac{C_c}{1+e_o}$ or $C_c = (1 + e_o)\ C_r'$. The quantity of C_r' is a linear function of e_o as shown in Appendix A. Therefore, the C_c is a function of $(C_r')^2$ which is verified by the relationship $C_c = 0.0844 + 9.121\ (C_r')^2$, as shown in Appendix A-IV.

The recompression index (C_r) is a rebound parameter of a clay soil. In practice it is usually taken as a fraction of C_c. In this study it is found that $C_r = A + B\ C_c$ where A is - 0.00327. The standard deviation of A is 0.00199, B is 0.139, and the standard deviation of B is 0.00726. Therefore, with 95% confidence the C_r will lie in the range of, approximately, $\frac{1}{6.5}\ C_c$ and $\frac{1}{8}\ C_c$ for Indiana soils.

As mentioned in Section 2.2.1.c the preconsolidation pressure (p_c) was correlated with liquidity index (LI). Figure 4.1 shows the scatter in this relationship. In the

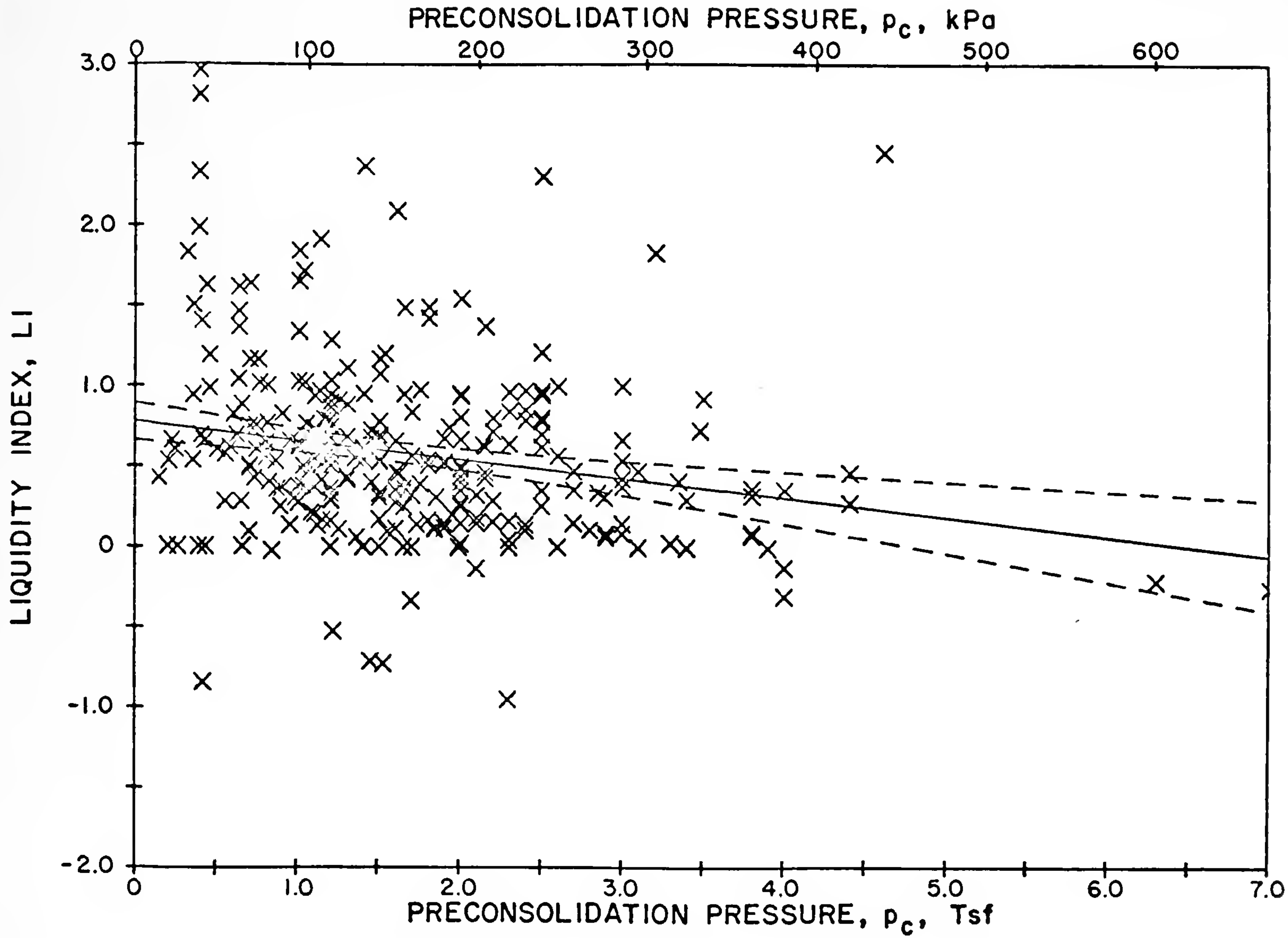


Figure 4-1 Preconsolidation Pressure vs Liquidity Index, Indiana Soils

presentation of the data, the solid line represents the best fit line while the dashed lines define the boundaries of 95% population confidence interval. Therefore,this correlation is not a strong one for Indiana soils. A regression equation was developed for p_c (Appendix A-IV), which shows the functional relationship of p_c versus moisture content, initial void ratio, and natural dry density. Again, the scatter in the data is quite large, as indicated by a large standard deviation of estimate and the scatter plot of measured p_c versus predicted p_c ($\hat{p}_c$) as shown in Figure 4-2. A part of the difficulty may lie in determining accurate preconsolidation pressures from consolidation tests conducted in the standard way.

4.3.c. Strength Parameters

As mentioned in Section 2.2.3.c there were many qualifications to the relationships between effective strength angle and cohesion intercept versus the plasticity characteristics. Figures 4-3 to 4-6 show no definite correlation of strength angle or cohesion intercept with plasticity characteristics for either unconsolidated undrained, consolidated undrained (unsaturated), or consolidated undrained (saturated) tests.

Figures 4-7 through 4-9 show that as the log of the unconfined compressive strength (q_u) increases, the natural dry density increases but the natural moisture content and the liquidity index decrease. In the

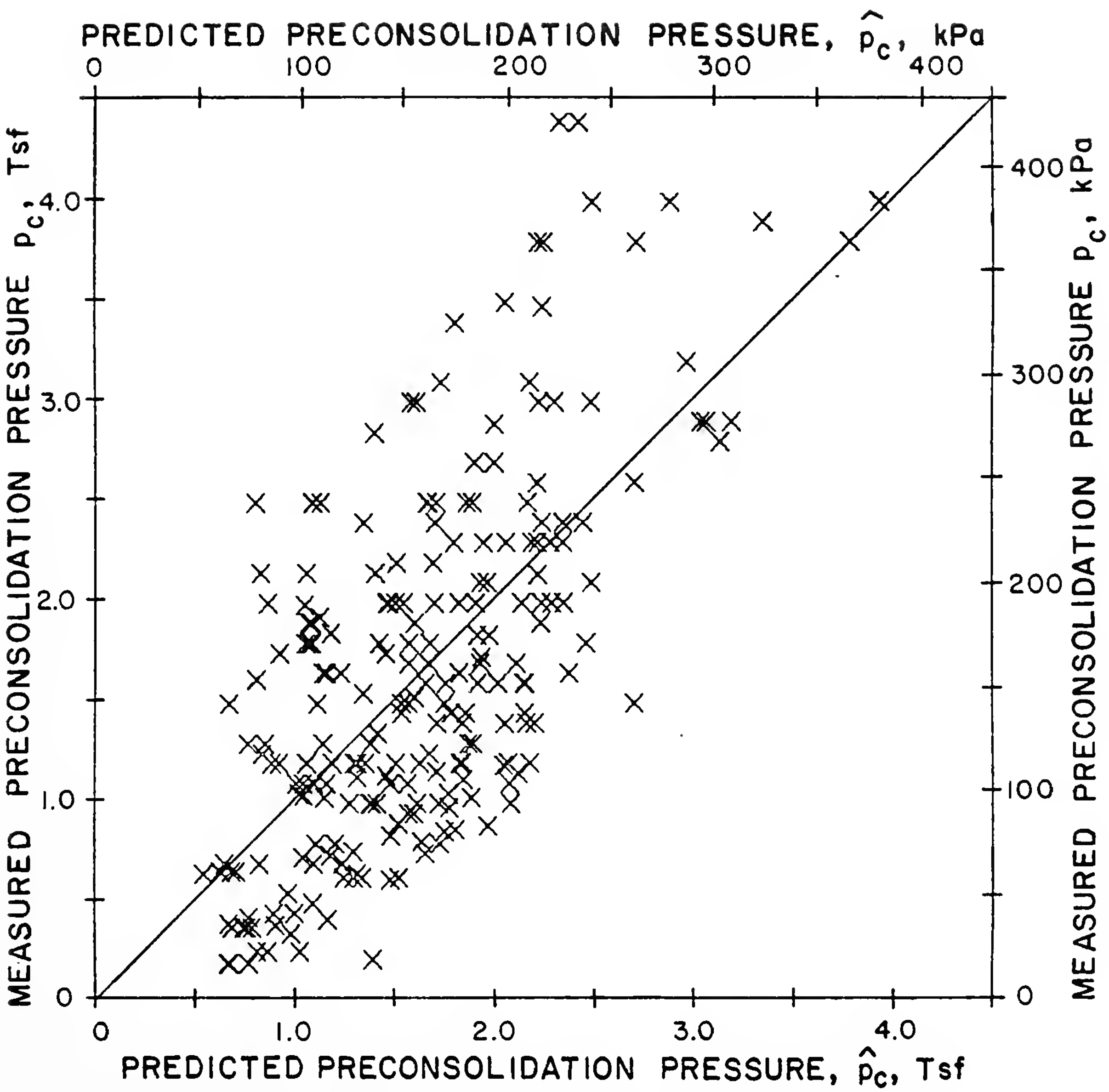
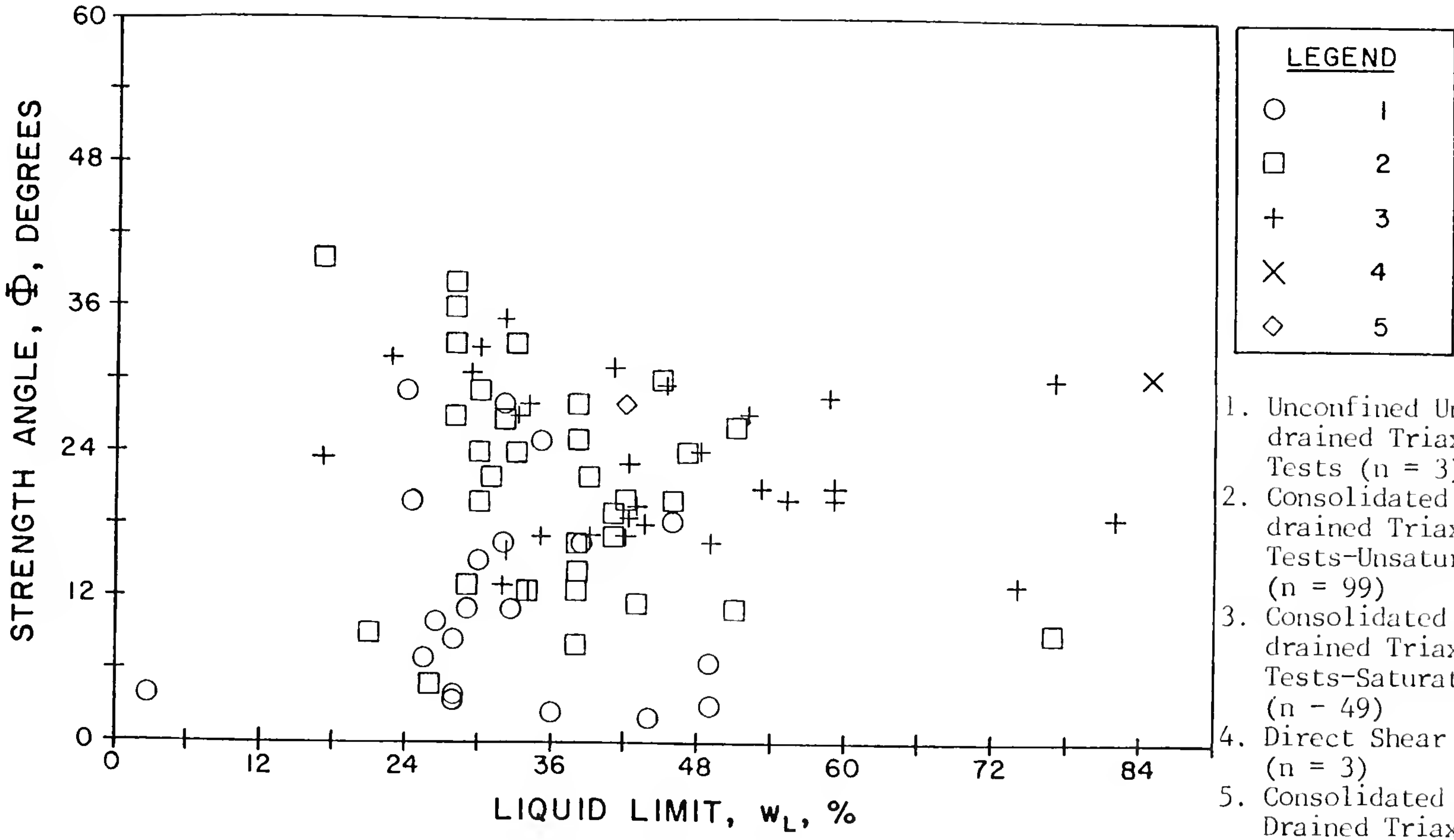


Figure 4-2 Comparison of Measured and Predicted Values of Preconsolidated Pressure (Data from Indiana Geotechnical Bank).



1. Unconfined Undrained Triaxial Tests (n = 3)
2. Consolidated Undrained Triaxial Tests-Unsaturated (n = 99)
3. Consolidated Undrained Triaxial Tests-Saturated (n - 49)
4. Direct Shear Tests (n = 3)
5. Consolidated Drained Triaxial Tests (n = 3)

Figure 4-3 Strength Angle vs Liquid Limit, Indiana Soils

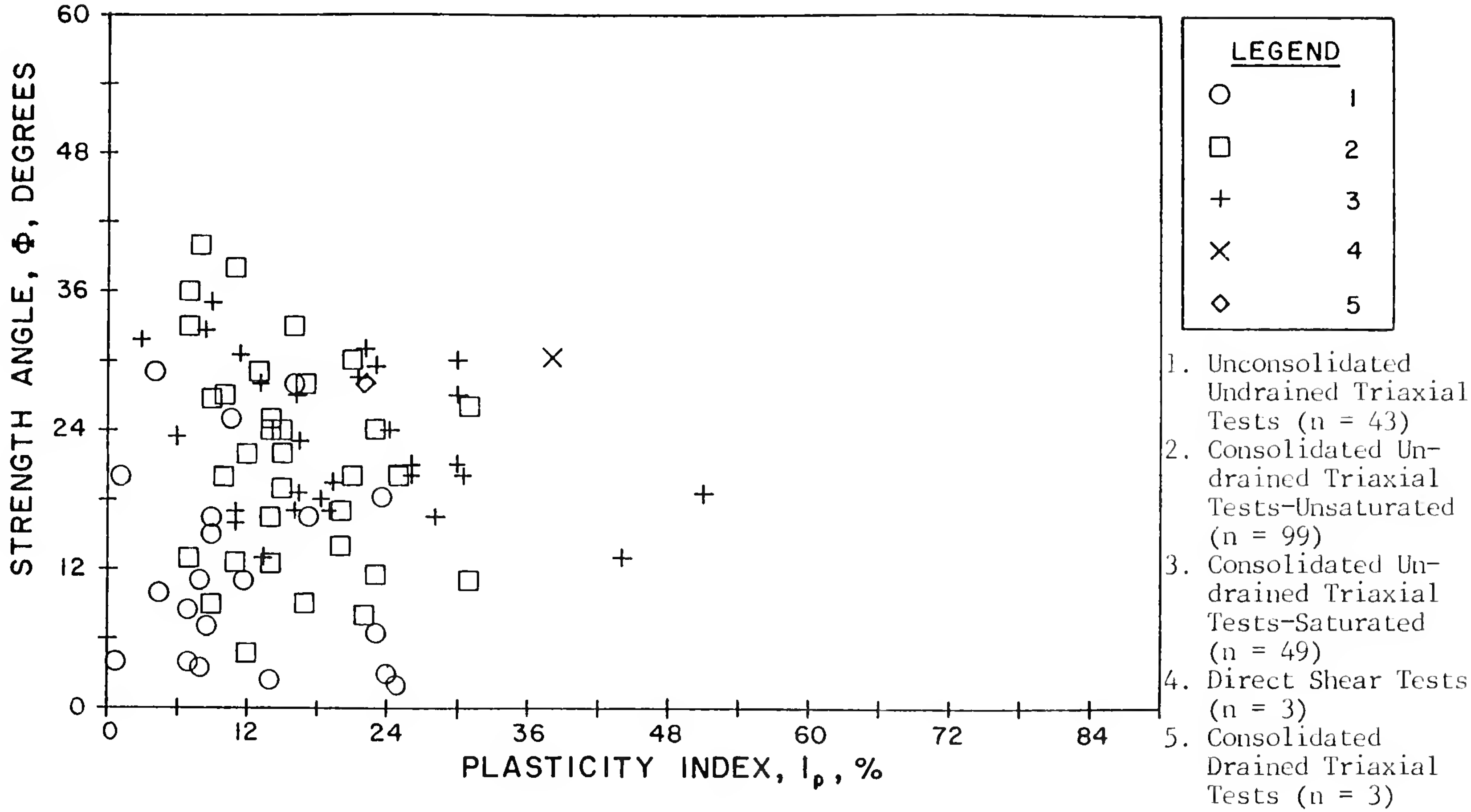
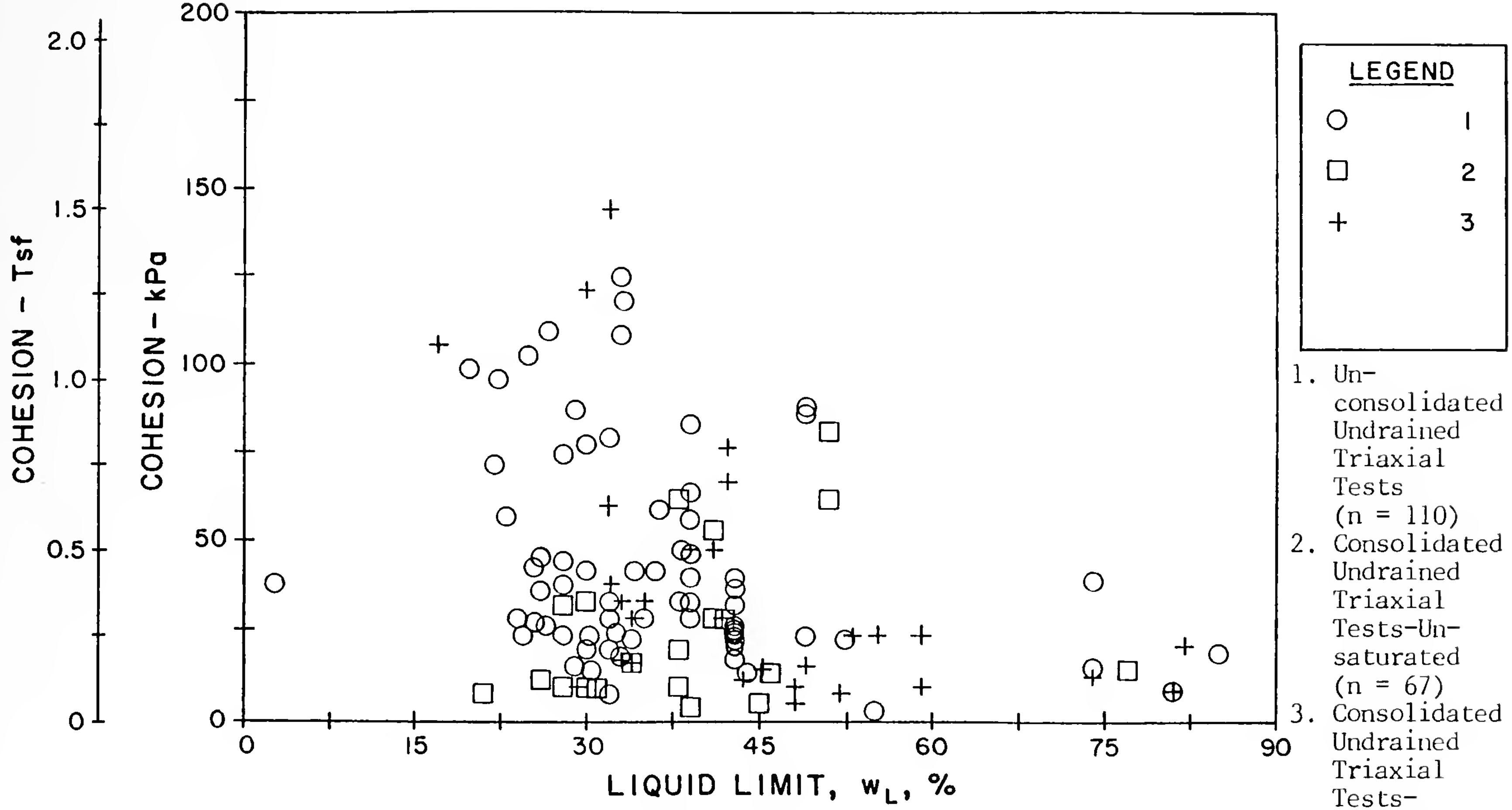


Figure 4-4 Strngth Angle vs Plasticity Index, Indiana Soils



1. Un-consolidated Undrained Triaxial Tests (n = 110)
2. Consolidated Undrained Triaxial Tests-Un-saturated (n = 67)
3. Consolidated Undrained Triaxial Tests-Saturated (n = 42)

Figure 4-5 Cohesion vs. Liquid Limit, Indiana Soils

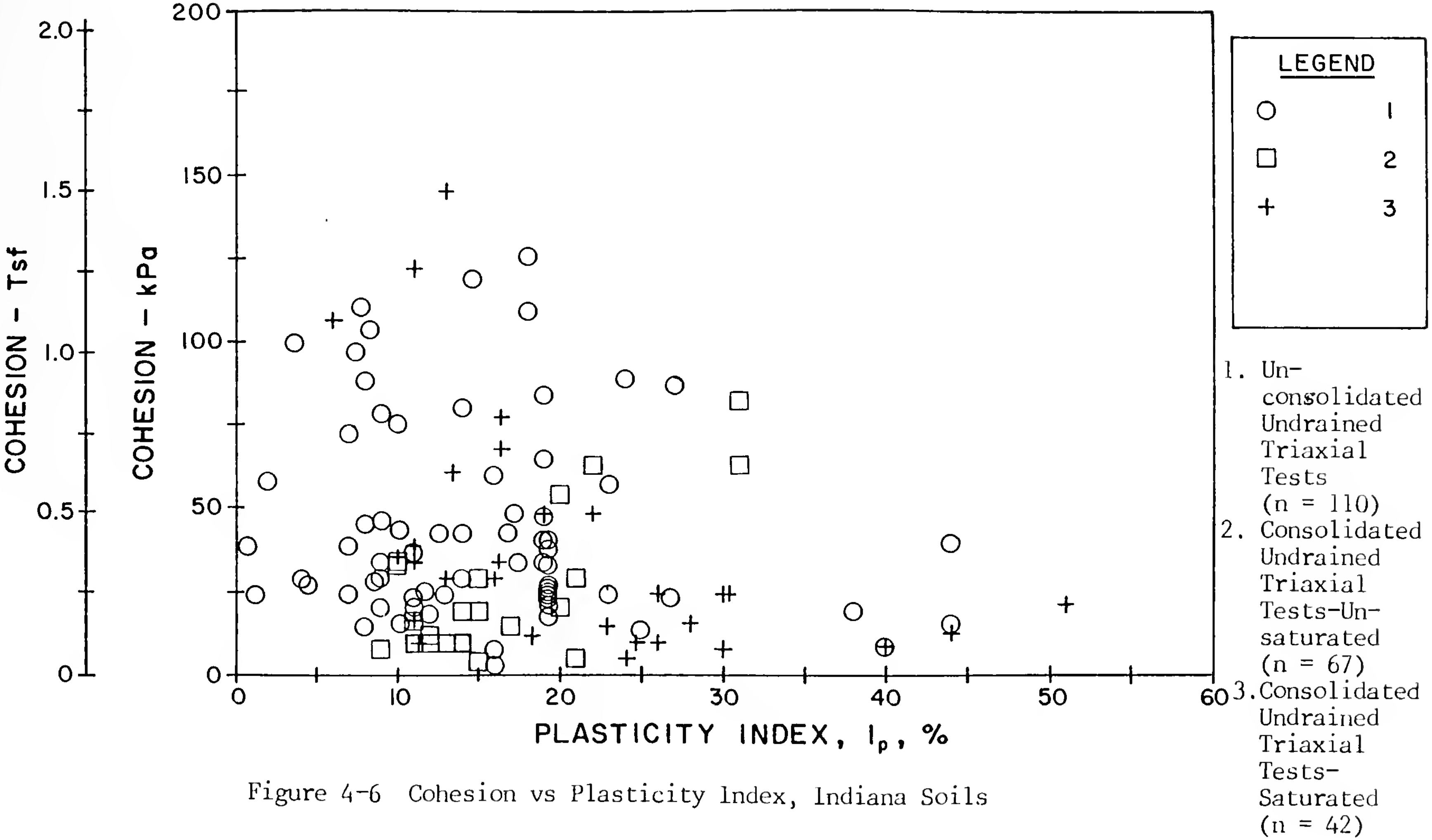


Figure 4-6 Cohesion vs Plasticity Index, Indiana Soils

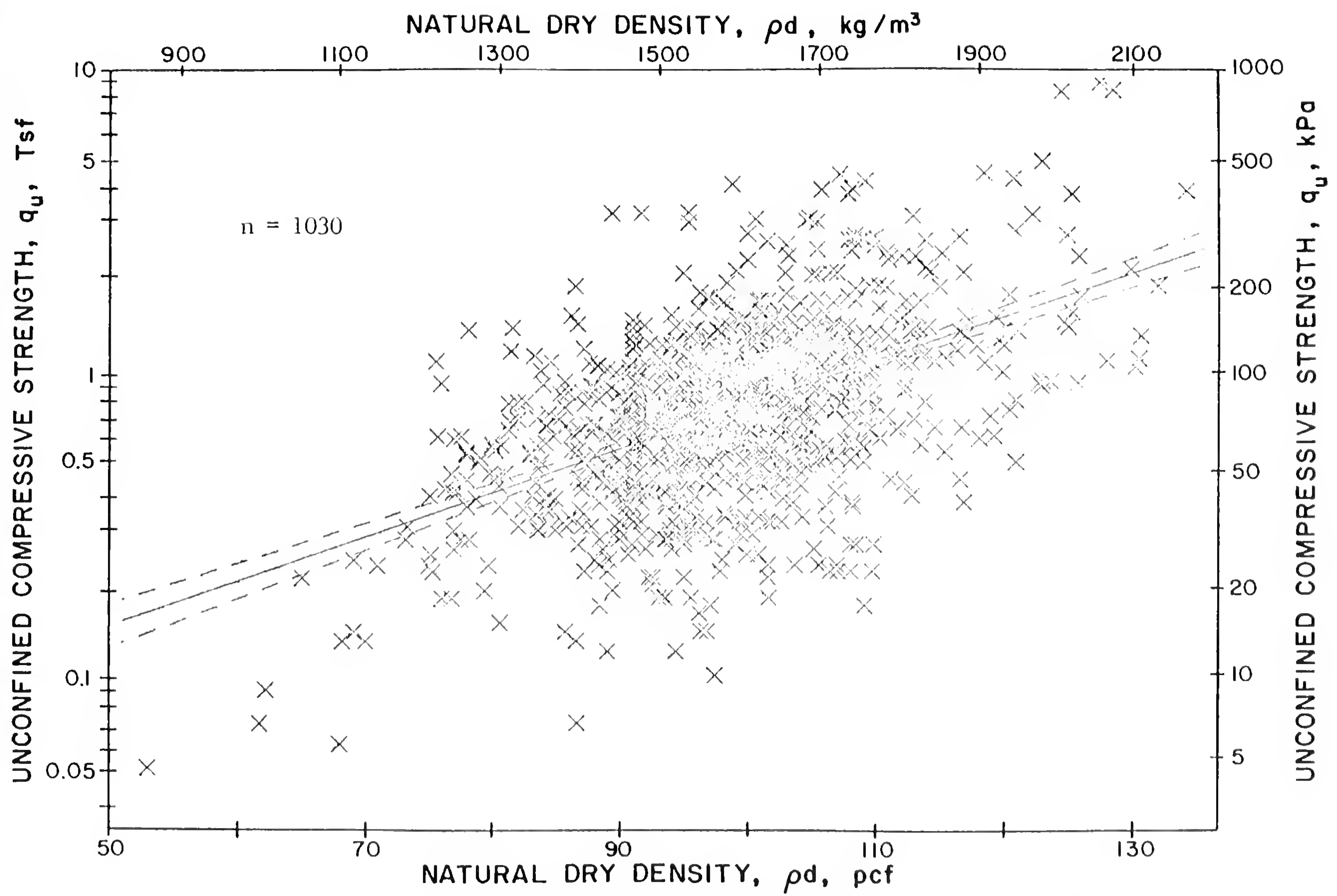


Figure 4-7 Unconfined Compresssive Strength vs Natural Dry Density, Indiana Soils

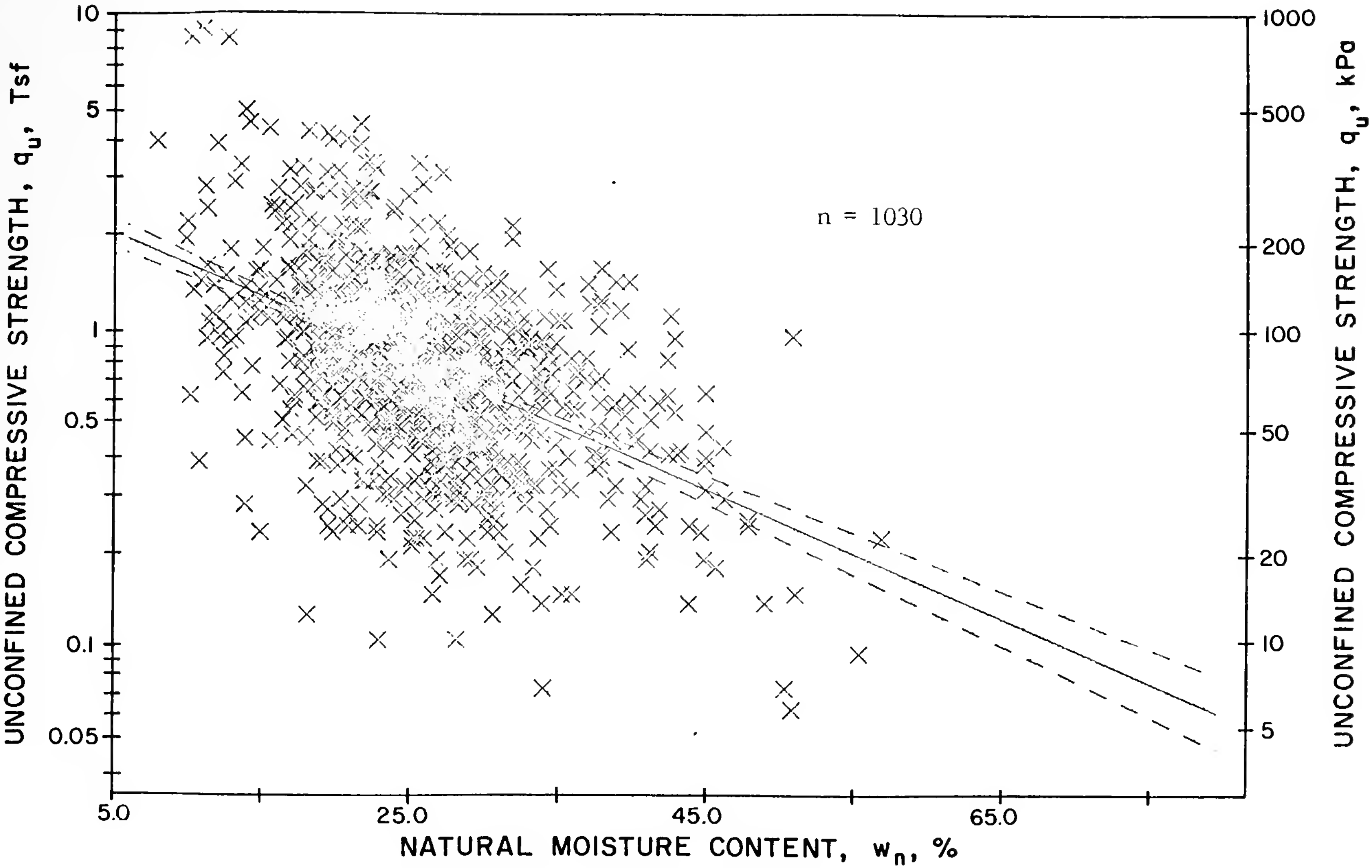


Figure 4-8 Unconfined Compressive Strength vs Natural Moisture Content, Indiana Soils

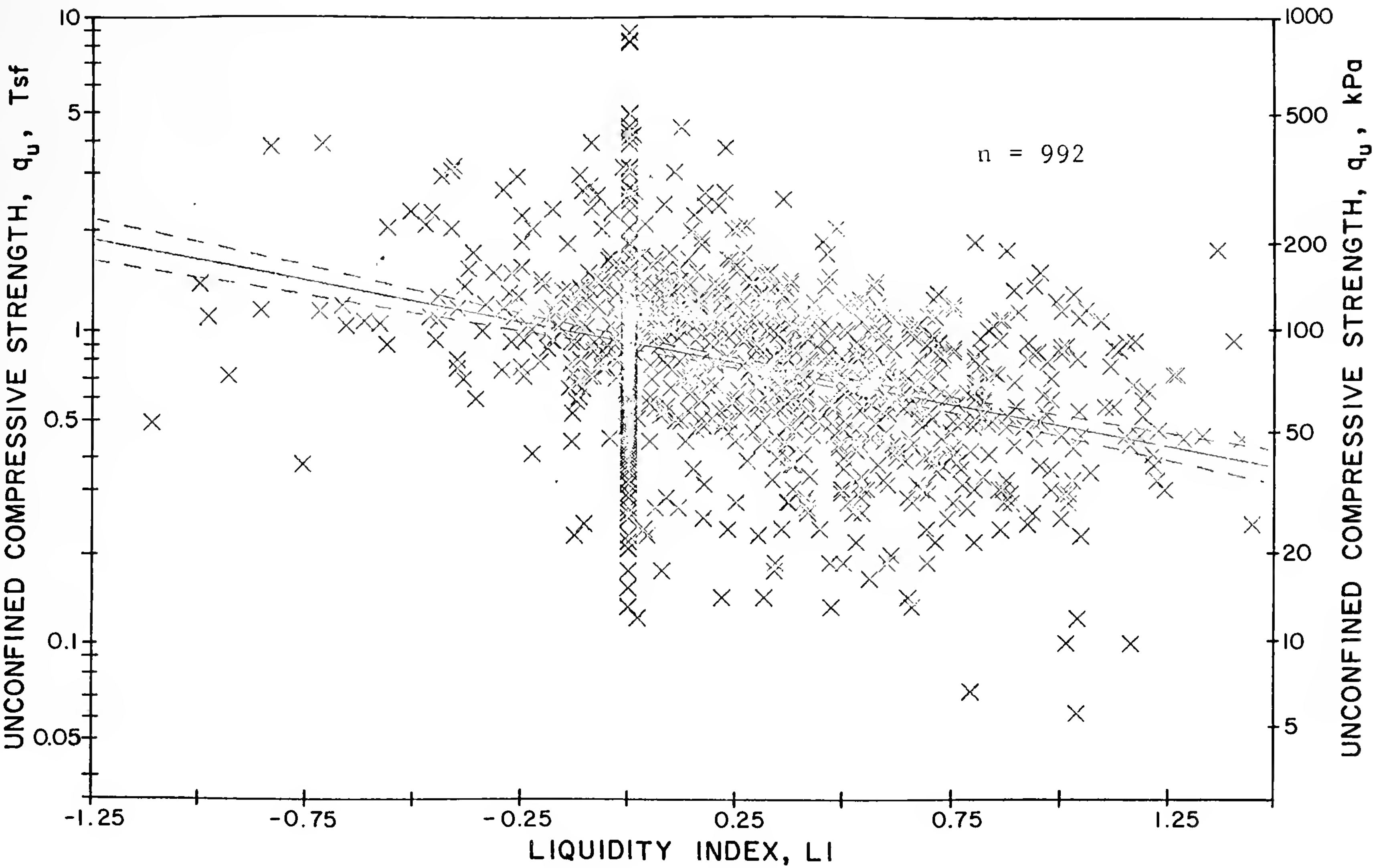


Figure 4-9 Unconfined Compressive Strength vs Liquidity Index, Indiana Soils

presentation of the data, the solid line represents the best fit line while the dashed lines define the boundary or 95% population confidence interval. Since the scatter in the data is quite large, these relationships are qualitative rather than quantitative.

The limit limit (w_L) is found to be a function of natural moisture content (w_n), SPT, and location factors, i.e., physiographic regions (x's) and parent material (z's) as shown in Appendix A. The equation may be approximately written as

$$
\begin{aligned}
w_L &= 48 + 0.50\ w_n - 10\ \log_{10}SPT + .050\ w_n\ \log_{10}SPT \\
&\quad + x's + z's \\
&= w_n - 0.50\ w_n - 10\ \log_{10}SPT + 0.50\ w_n\ \log_{10}SPT + 48 \\
&\quad + x's + z's \\
&= w_n - 0.50\ w_n\ \log_{10}10 + 0.50\ w_n \log_{10}SPT \\
&\quad - \log_{10}SPT + \log_{10}10 + 38 + x's + z's \\
&= w_n + 0.50\ w_n\ (\log_{10}SPT - \log_{10}10) - 10\ (\log_{10}SPT \\
&\quad - \log_{10}10) + 38 + x's + z's \\
&= w_n + (0.50\ w_n - 10)\ (\log_{10} \frac{SPT}{10}) + 38 + x's + z's.
\end{aligned}
$$

It can be seen that as the natural moisture content (w_n) = 20(%), the liquid limit of the soil is simply equal to 58 + x's + z's. For example, for the association Miami-Russell-Fincastle in Tippecanoe County, x_1 (Tipton Till Plain) = 1, $x_2 = x_3 = \ldots = x_{11} = 0$, $z_8 = 1$, and $z_1 = \ldots = z_7 = z_9 = \ldots = z_{11} = 0$, $w_L = 58 - 19.323 + 23.069 = 61.746$(%) at $w_n = 20$(%), 68(%) of the samples lie within 61.746 - 19.503 = 42.243 (%) and 61.746 + 19.503 = 81.249(%).

Figures 4-10 and 4-11 show the plots of unconfined compressive strength (q_u) versus SPT and the strength angle (ϕ) versus SPT, respectively. In the presentation of the data, the solid line represents the best fit line while the dashed lines define the boundaries of 95% population confidence interval. Since the scatter is large, no reliable predictions can be made.

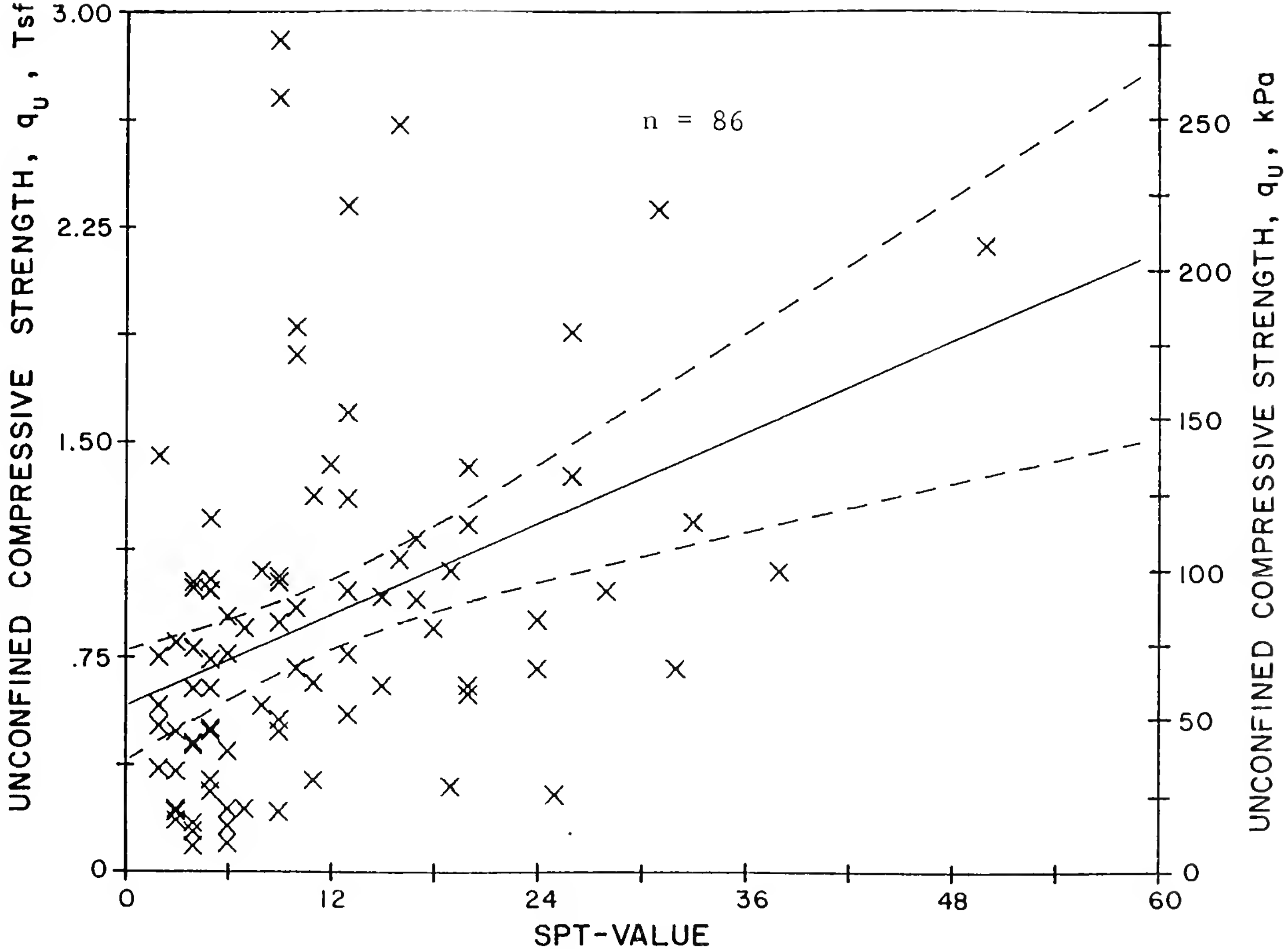


Figure 4-10 Unconfined Compressive Strength vs SPT, Indiana Soils

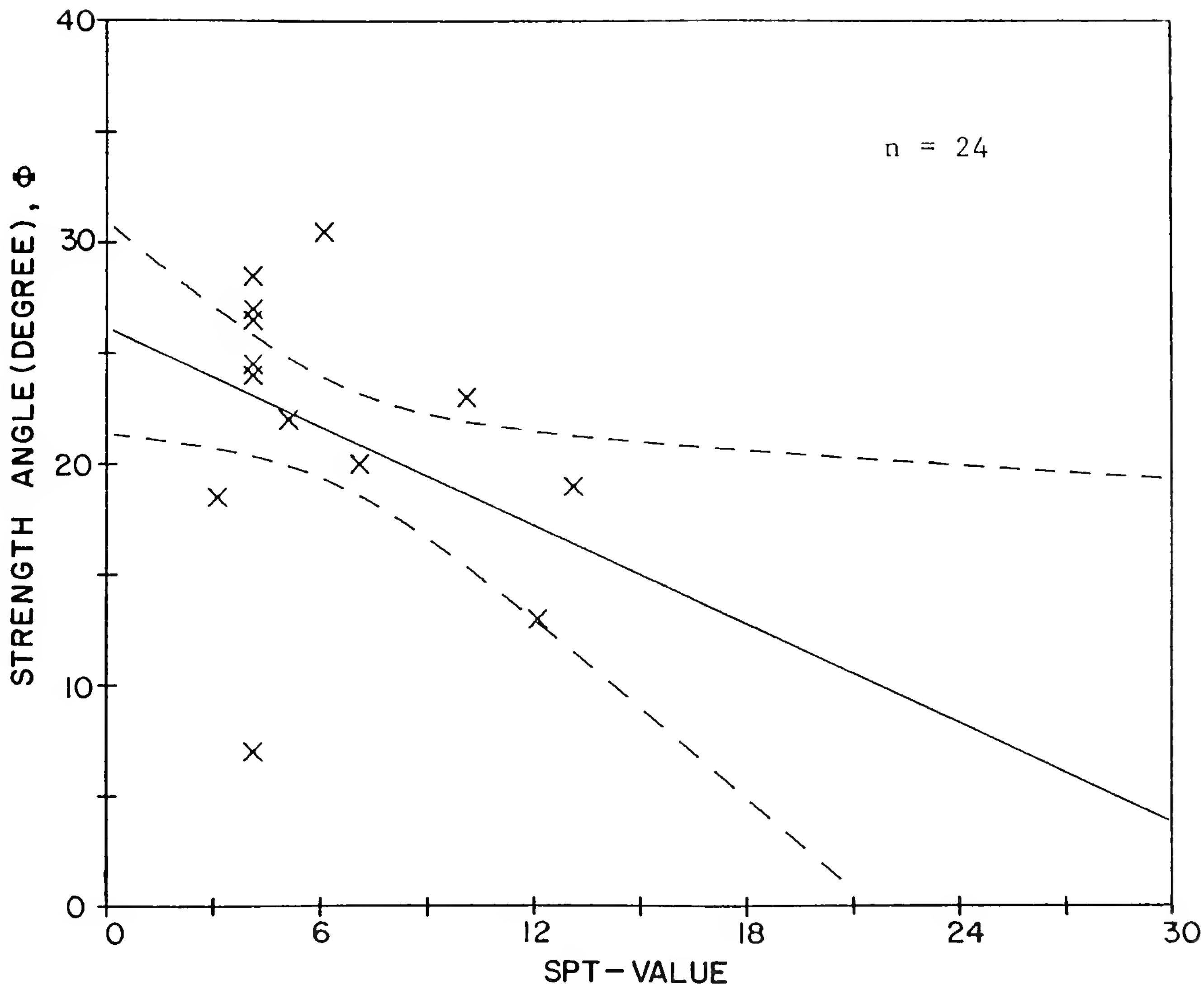


Figure 4-11 Strength Angle vs SPT, Indiana Soils

CHAPTER 5 APPLICATIONS

5.1 Topographic Features

It is important to define the general topographic features of the area of interest before attempting any preliminary design. The topographic features include the ground elevation, water depth with relation to ground elevation, slope, and the depth to bedrock. Some of these can be inferred from the statistical soil profiles (Appendix A-III). However, due to the variation of topography with location either routine data retrieval or reference to the Soil Survey Manuals (107) may be more helpful.

5.2 Shallow Foundations

The design of a foundation unit usually requires that both bearing capacity and settlement be checked.

5.2.a Bearing Capacity

In order to determine the bearing capacity of both cohesive (silty clay, clay) and cohesionless (sand) soils the strength parameters within the depth of influence of the footing are required. The general bearing capacity equation contains the variables of size of the footing, density of the soil, depth of the footing, strength angle, and cohesion of the soil. The density, strength angle,

and cohesion can be estimated either directly from the statistical soil profiles or from the routine retrieval of data.

For many locations in the state, the ground water table is at a shallow depth, viz., a few feet, as shown in Appendix A-I and A-III. Therefore, the soils may often be assumed to be saturated. For cohesive soils the undrained shear strength, $c_u = \frac{1}{2} q_u$ (unconfined compressive strength), may be used. The q_u can be obtained either from the appropriate statistical soil profiles, from the regression equation based upon the index properties, as shown in Appendix A-IV, or from routine retrieval of data.

The allowable bearing pressure in sands can be roughly estimated by using the SPT results. For details of the procedures refer to Peck, et al (80). The distribution of SPT versus depth for the area of interest can be obtained either from the statistical soil profiles or from data retrieval.

5.2.b Settlement

The consolidation settlement of cohesive soil is normally calculated using the following soil parameters: the initial void ratio, overburden pressure, compressive index, recompression index, and preconsolidation pressure. The initial void ratio and the overburden pressure are relatively easy to measure or estimate. The other soil parameters can be evaluated by using the regression

equations shown in Appendix A-IV. The preconsolidation pressure can also be roughly estimated from either the statistical soil profiles or from direct data retrieval.

The settlement of cohesionless soil can be estimated by empirically using SPT results. For details refer to Terzaghi and Peck (113), Meyerhof (64) and Peck et al (80).

5.3 Pile Foundations

The depth to bedrock can be retrieved from the data bank or estimated by consulting Soil Survey Manuals (107) for the area of interest.

The geometry of the pile, SPT values along the pile, density of the soil, and the strength angle which is used to determine the bearing capacity coefficient are required to predict the ultimate bearing capacity of a single pile in granular soils by the so-called "theoretical method" (64, 74).

The ultimate bearing capacity of a single pile in clay can be estimated by the size of the pile shaft and the adhesion strength, which is correlated with the undrained shear strength of the soil (74).

The density, SPT, strength angle, and the undrained shear strength (half of unconfined compressive strength) can be obtained from either the correlations presented in this work or by data retrieval.

5.4 Cut Slopes

In addition to the information of topography and soil identification, the strength parameters, such as the unconfined compressive strength, cohesion, and strength angle, are required for the analyses of slope stability. A variety of methods can then be applied to the problems using proper sets of strength parameters. For details of these methods refer to Taylor (111), Terzaghi and Peck (113), and Chowdhury (19).

5.5 Excavations and Retaining Structures

The active and passive earth pressure coefficients may be estimated by using the geometry of backfills and retaining structures, and the appropriate strength parameters. For details of analyses refer the NRCC (74), Tschebotarioff (117), and U.S. Navy (118).

5.6 Compaction Requirements for Embankments

The index properties of soils can be used as guidelines for the suitability of excavation or borrow materials. The control compaction parameters, such as maximum dry density and optimum moisture content, are needed for the design and construction of embankments. They can be evaluated for a given soil by using the existing results and regression equations as shown in Appendices A-II, A-III, and A-IV.

5.7 Pavements

For the design of roadway pavement the existing results as represented in this work and in the data bank can provide the following information: the topography of the area of interest, soil classification, compaction parameters, i.e., maximum dry density, optimum moisture content, and the soaked CBR values of the subgrade material at either 100% of maximum dry density or 95% maximum dry density. With this information, the corresponding R-value, bearing value, and the modulus of subgrade reaction can be estimated by using the chart developed by the Portland Cement Association (PCA) (83). Providing the traffic information is known, either the flexible (asphalt) or the rigid (concrete) pavement can be designed.

5.8 Example 1

The following example is to illustrate the uses of the data bank for a typical highway project.

5.8.1 Project Identification

There is to be a new route along the old S.R. 912 located from East Chicago, Lake County, Indiana to Chicago, Illinois in the area of R9W and T37N. The geotechnical works involve: one bridge, with five spans ranging in length from 90 ft. to 100 ft., one embankment of 30 ft. height, and 5 retaining walls to be constructed along the route. The length of the highway is approximately 3.5 miles. Ten cuts are also along the route. It is desirable to have the

geotechnical information in this area for preliminary considerations.

5.8.2 General Scope of Procedures

The use of the Indiana geotechnical data bank aids in the preliminary phases of design and construction of the project. To accomplish this purpose,the investigation is divided in the following parts:

(1) determination of the soil types and topography within the area;

(2) determination of the strength and other engineering characteristics of these soils;

(3) recommendations to aid the subsurface investigation program;

(4) recommendations to aid design and construction of the proposed highway.

5.8.3 Determination of the Soil Types

Physiographically, the site is located within the Calumet Lacustrine Plain of the Northern Lake and Moraine Region. The soil association is Oakville-Plainfield-Adrian. The distributions of ground elevation and the water depth with relation to ground elevation were examined. It was found that the mean of the ground elevations was 588 ft., its s.d. was 3.87 ft.; the median was 589 ft. and I.R. was 3.55 ft. Therefore, no appreciable topographic relief was found. The ground water elevation was rather level with median of 6.20 ft. and I.R. of 2.5 ft. According to

the geological maps prepared by the Indiana Geological Survey, the bedrock (limestone) is generally at depths ranging between 100 and 150 ft.

The distributions of textures of soils versus depth for each section along the route were then examined. The subsurface investigation revealed a relatively uniform soil profile. Beneath the ground surface there existed a sandy deposit approximately 32 feet thick with occasional silty sand deposits and gravelly channels at random locations. Surface drainage at this area was good due to the granular nature of the soils.

Beneath the sand, starting at approximately El 557 was a stiff plastic clay. This clay extended approximately 55 ft. (to El 502) and was underlain by a much stiffer and less plastic clay (hard pan) (Figures 5-1 to 5-4).

5.8.4 Determination of the Engineering and Strength Characteristics

The data search program was next directed toward the determination of the engineering and strength characteristics of the various soils encountered in this project. Results are illustrated in Figures 5-1 to 5-4 and tabulated in Table 5-1 Further details of the computer programs have been included in the Appendix B-2.

The results of consolidation tests for clay samples were examined. No substantial preconsolidation is apparent. The clay exhibits a decrease in plasticity with depth, with

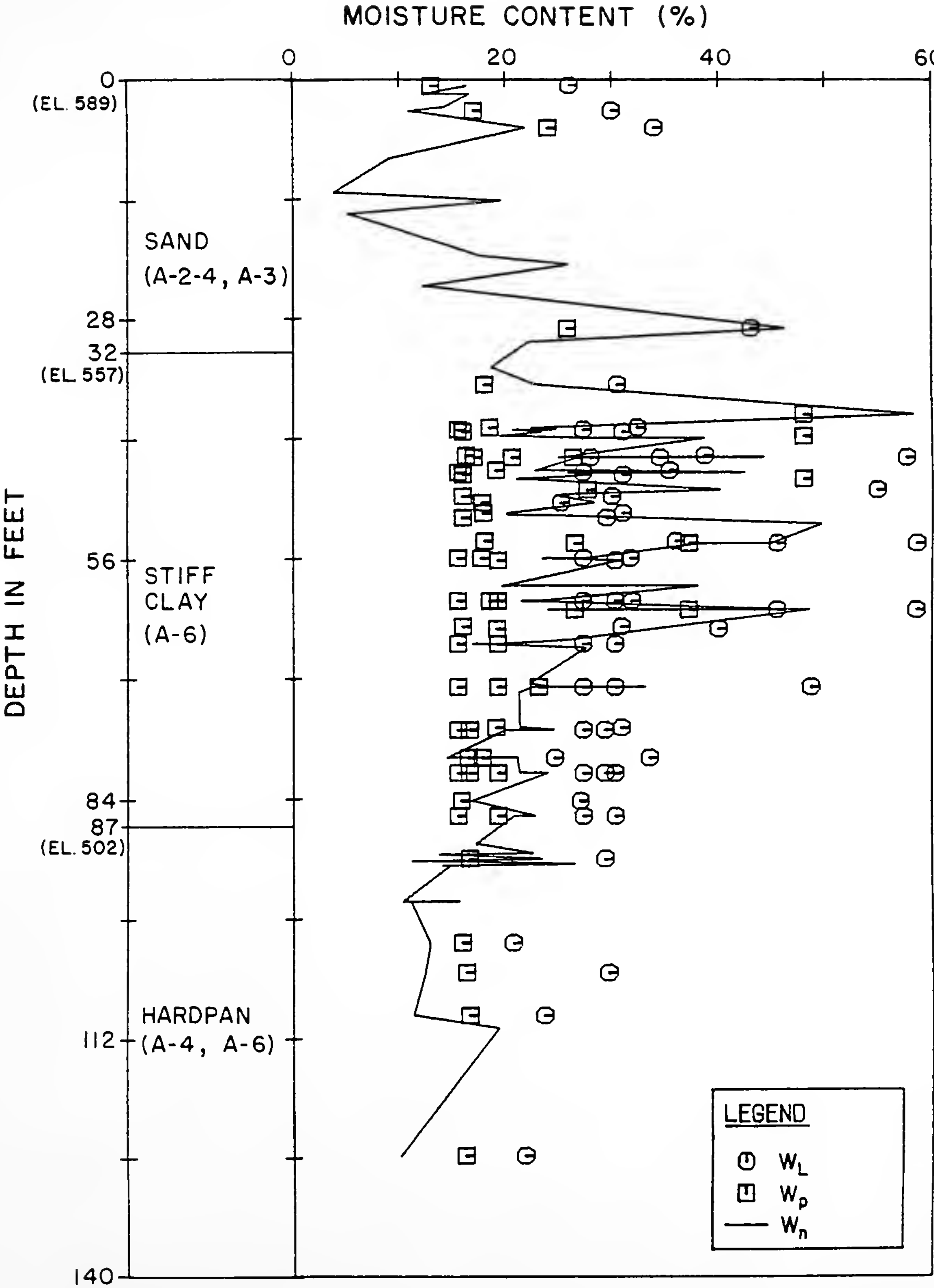


Figure 5-1 Water Content vs Depth, East Chicago (Data from Indiana Geotechnical Data Bank)

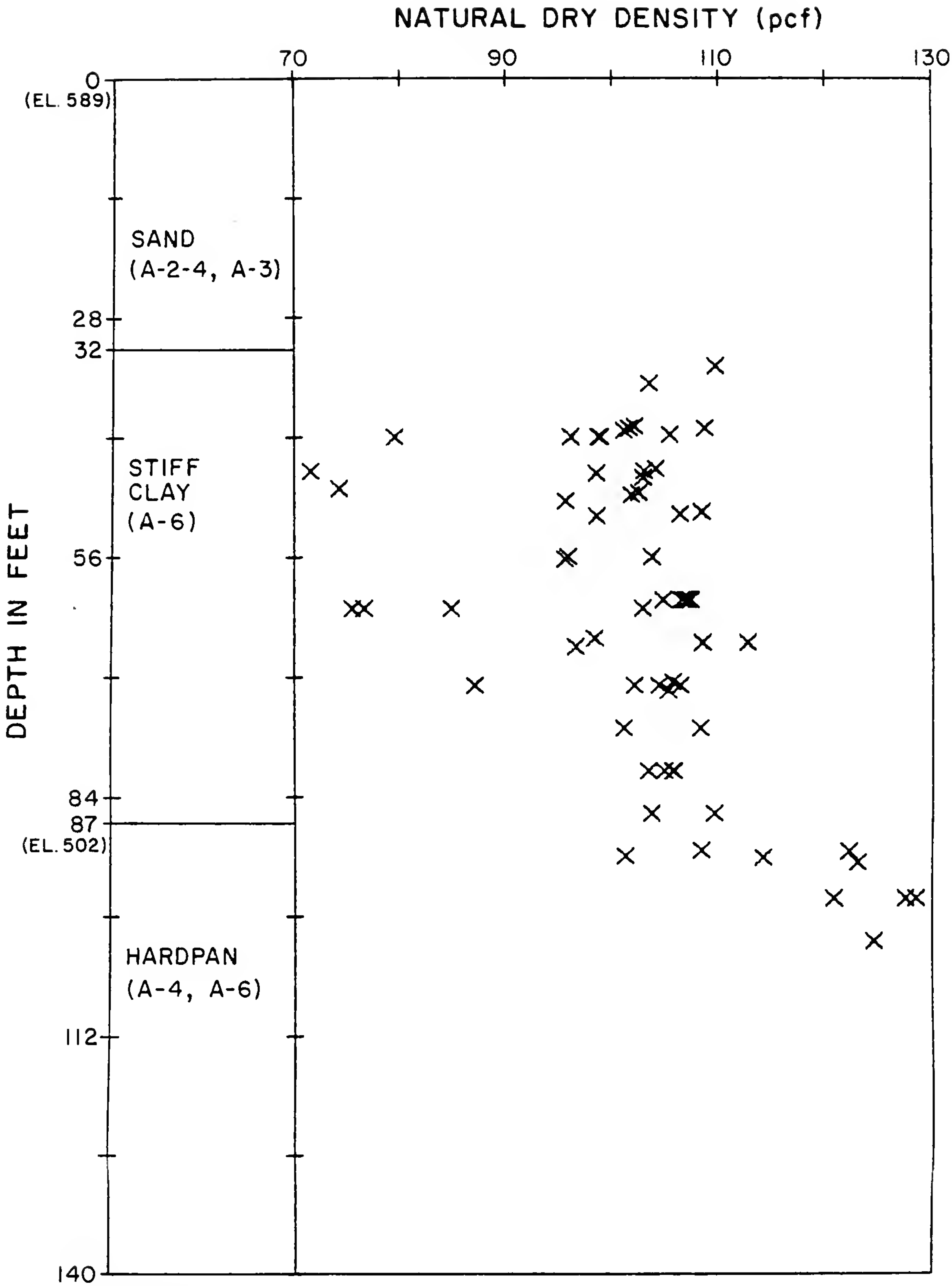


Figure 5-2 Natural Dry Density vs Depth, East Chicago, (Data from Indiana Geotechnical Data Bank)

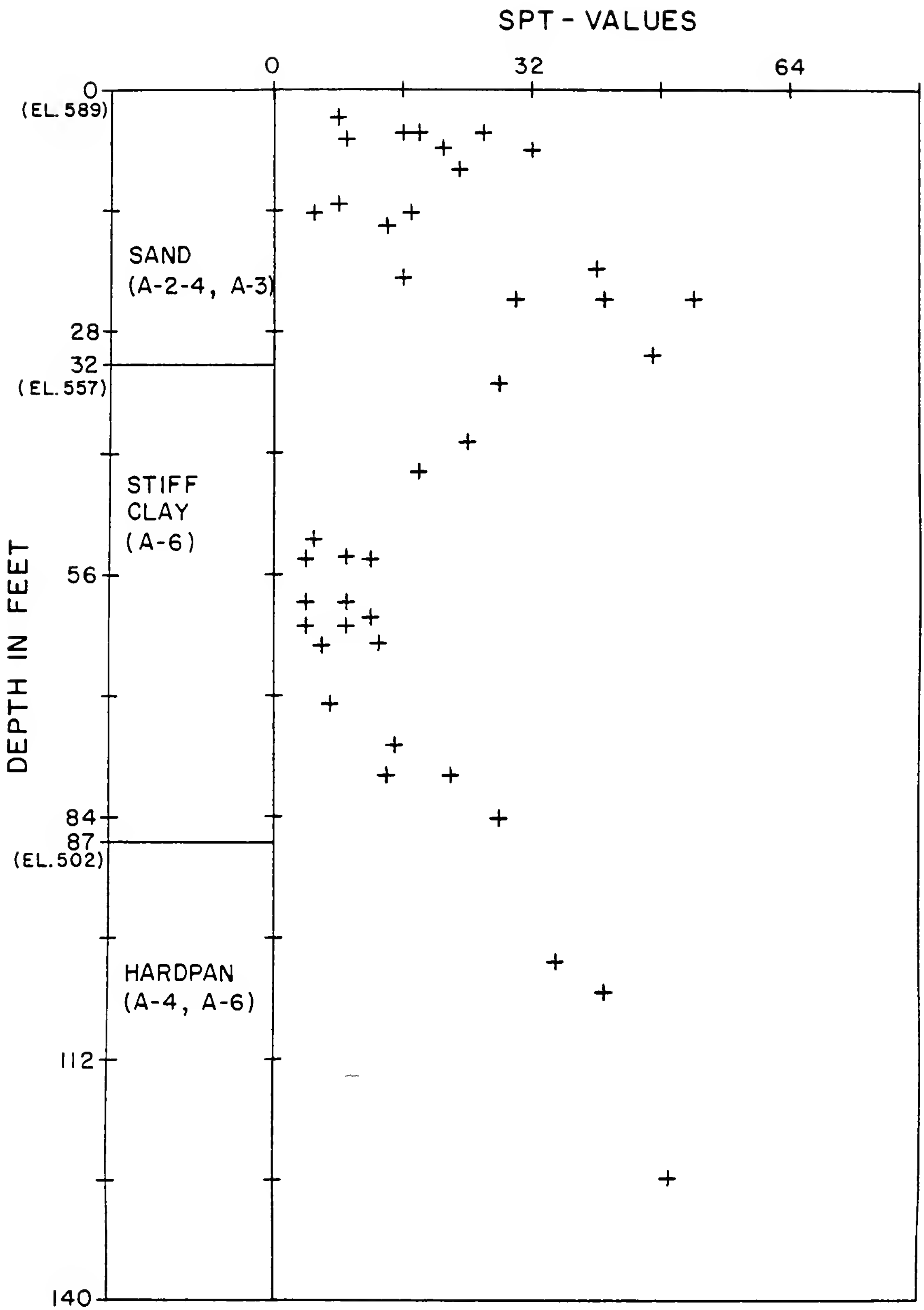


Figure 5-3 SPT vs Depth, East Chicago, (Data from Indiana Geotechnical Data Bank)

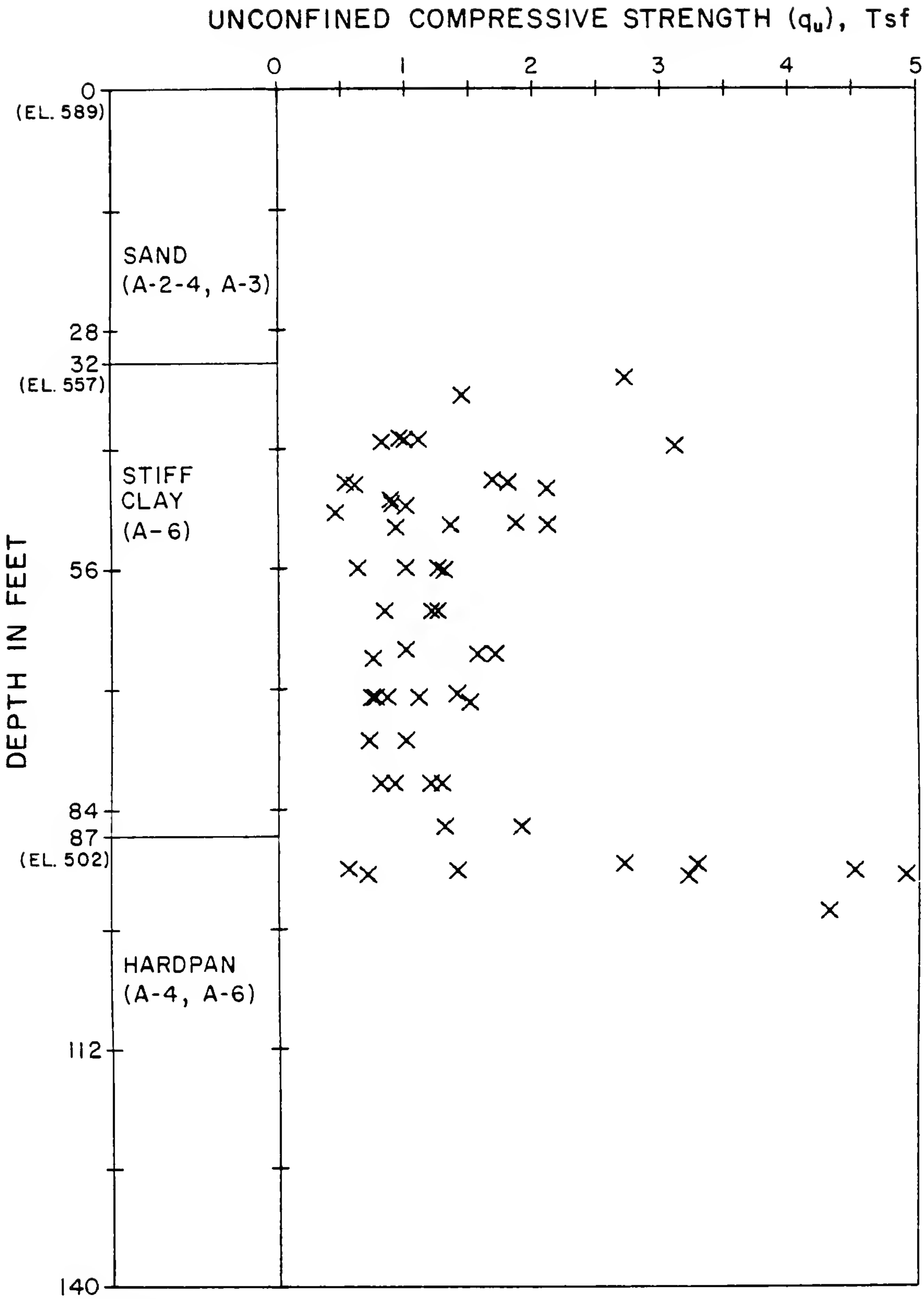


Figure 5-4 Unconfined Compressive Strength vs Depth, East Chicago (Data from Indiana Geotechnical Data Bank)

Table 5-1. Engineering and Strength Characteristics of East Chicago Soils

Depth	G	$\rho_{d\ max}$	OMC	CBR SO1	CBR SO2	e_o	C_c	C_r	c_v	Type	c	ϕ
2.00	2.700	100.7	16.8	31.5	19.2	-0	-0	-0	-0	-0	-0	-0
3.25	-0	99.5	15.0	-0	-0	-0	-0	-0	-0	-0	-0	-0
3.50	-0	102.8	13.7	-0	-0	-0	-0	-0	-0	-0	-0	-0
35.75	-0	-0	-0	-0	-0	.569	.144	.030	-0	-0	-0	-0
40.75	-0	-0	-0	-0	-0	.572	.149	.030	-0	-0	-0	-0
42.00	-0	-0	-0	-0	-0	-0	-0	-0	-0	3.	.05	24.5
42.00	-0	-0	-0	-0	-0	-0	-0	-0	-0	3.	.05	24.5
42.00	-0	-0	-0	-0	-0	-0	-0	-0	-0	3.	.05	24.5
44.00	2.710	-0	-0	-0	-0	.704	.195	.037	1.47	-0	-0	-0
46.00	2.740	-0	-0	-0	-0	.659	.199	.024	-0	-0	-0	-0
46.25	2.740	-0	-0	-0	-0	.729	.238	.037	-0	-0	-0	-0
48.00	2.670	-0	-0	-0	-0	1.392	.590	.130	-0	-0	-0	-0
50.75	-0	-0	-0	-0	-0	.573	.151	.030	-0	-0	-0	-0
51.25	2.740	-0	-0	-0	-0	.739	.220	.037	-0	-0	-0	-0
54.00	2.710	-0	-0	-0	-0	.736	.170	.029	1.47	-0	-0	-0
56.00	2.760	-0	-0	-0	-0	.706	.190	.050	4.26	-0	-0	-0
56.25	2.740	-0	-0	-0	-0	.785	.244	.077	-0	-0	-0	-0

Table 5-1. (continued)

Depth	G	$\rho_{d\ max}$	OMC	CBR SO1	CBR SO2	e_o	C_c	C_r	c_v	Type	c	ϕ
62.00	2.790	-0	-0	-0	-0	.693	.175	.030	-0	-0	-0	-0
62.00	2.698	-0	-0	-0	-0	1.240	.280	.035	-0	3.	0	28.5
62.00	2.690	-0	-0	-0	-0	1.240	.280	.035	-0	3.	0	28.5
62.00	2.690	-0	-0	-0	-0	1.240	.280	.035	-0	3.	0	28.5
64.00	2.720	-0	-0	-0	-0	.630	.210	.018	1.47	-0	-0	-0
71.00	2.740	-0	-0	-0	-0	1.115	.380	.130	-0	-0	-0	-0
76.00	2.740	-0	-0	-0	-0	.690	.173	.099	-0	-0	-0	-0

Note:
1. (-0) Unknown.
2. (Type 3) Consolidated undrained triaxial test - saturated.
3. Depth - depth to ground surface in ft.
   G - specific gravity.
   ρ_{dmax} - maximum dry density (Pcf).
   OMC - optimum moisture content (%).
   CBR SO1 - CBR soaked 100% $\rho_{d\ max}$ (%).
   CBR SO2 - CBR soaked 95% $\rho_{d\ max}$ (%).
   e_o - initial void ratio
   C_c - compression index.
   C_r - recompression index.
   c_v - coefficient of consolidation (ft^2/mth).
   c - cohesion (Tsf)
   ϕ - strength angle (degree)

a sharp drop in plasticity at approximately elevation 502 feet (Figure 5-1).

5.8.5 Recommendations for Preliminary Design

5.8.5.a <u>Highway Embankment.</u> For the areas where the highway or access ramps will be on fill, standard design and construction procedures, which include compaction requirements, topsoil stripping, and undercutting existing loose fill, are considered satisfactory. It is recommended that standard embankment slopes be used for the project. The embankment slopes will not be steeper than 3 horizontal to 1 vertical (except at end bents where the slopes will be 2 horizontal to 1 vertical).

In some areas, the fill may be retained by retaining walls. The recommendations with respect to design of the retaining walls are discussed in Section 5.8.5.d.

The values of compaction parameters listed in Table 5-1 are recommended for preliminary design. It is predicted that ρ_{dmax} = 101 pcf, and OMC = 15%.

5.8.5.b <u>Settlement of Embankments.</u> Due to the lack of apparent preconsolidation of the foundation clay layer, settlement analysis should be performed on the assumption that it is normally consolidated.

It is recommended that the 55 ft. thick compressible stratum from 32 ft. to 87 ft. be divided into three layers: e_o = 0.680, C_c = 0.197, C_r = 0.031, and c_v = 1.47 ft^2/mth for the layer from 32 ft. to 50 ft.; e_o = 0.785, C_c =

0.244, C_r = 0.077, and c_v = 1.47 ft^2/mth for the layer from 50 ft. to 68 ft.; and e_o = 0.690, C_c = 0.173, C_r = 0.099, and c_v = 1.47 ft^2/mth for the layer from 68 ft. to 87 ft.

5.8.5.c Bridge Foundations. Both possibilities of using shallow spread footings and deep foundations should be studied. If excessive total settlement and/or intolerable differential settlement are found for shallow spread footings, the deep foundations are recommended. Consideration of scour may also require deep foundations. Piles bearing in the hardpan encountered at about El 492 are recommended. If the end bent of the bridge foundation is also part of the retaining wall supporting the fill, the piles should be sleeved at least down to El 502 to minimize downdrag from negative skin friction resulting from the fill settlements. If the shallow spread footings are applicable, the base should be located at a depth at least 3.5 ft. below final grade for frost protection.

The information in Figures 5-1 to 5-4 and Table 5-1 serves the purpose of preliminary design. The strength angle is predicted to be 24^{o} and the strength intercept is 100 psf.

5.8.5.d Retaining Walls. It is not known what soils will be used for backfill. However, it is assumed that the backfill soils will be clean granular materials. The base of the retaining wall should be located at a depth of at least 3.5 ft. below final grade for frost protection. The

base strength angle is predicted to be 24^{o} and the cohesion is 100 psf.

5.8.5.e Ground Water. The ground water elevation appeared to be deep enough to permit footing bottoms to be located below frost depth (3.5 feet) without going below the water table. If the ground water rises (seasonal variation), the footing can be raised to avoid dewatering if an earth berm is placed for the minimum 3.5 ft. of frost cover. However, if the footing must bear below the water table for structural reasons, suitable footing drains must be employed.

5.8.5.f Piles, Corrosion. All pH tests indicated that the soils in this area are not corrosive to concrete and steel.

5.8.5.g Others. The CBR values in Table 5-1 seems high. It is recommended that more extensive CBR testing be performed for subgrade design. If piles are to be used for bridge piers, it is recommended that several pile load tests at various locations along the bridge be performed to determine the suitable pile capacities, as well as the depths to which piles must be driven to attain the desired capacity.

5.8.6 Recommendations for Subsurface Investigation Program

The data bank is not able to precisely locate the position in which the samples have been taken in the previous projects. The smallest location unit is the section (one square mile in area). Therefore, only general

recommendations can be made by using data bank as follows.

5.8.6.a Recommendations by Using Data Bank

1. The standard drilling equipment for making borings is assumed to be used for this project. The standard penetration test is recommended for sand and clay. The Shelby tube is recommended for sampling clay.
2. The depths of the standard penetration tests for the designs of bridge and wall foundations are suggested to be 150 ft. and 32 ft. respectively. Eleven standard penetration tests are required as a minimum. Four intact samples at the depth of 16 ft. beneath ground surface at each bridge pier site and at the depth of 10 ft. beneath ground surface at each retaining wall site are recommended.
3. Shelby tubes are to be used to obtain undisturbed clay samples at the site of embankment at the depths of, approximately 41 ft., 59 ft. and 78 ft. The depth of the boring is 145 ft. (hard pan). Five samples are collected for each specific depth.
4. The pH tests are recommended to be performed on samples at the depths of, approximately, 16 ft., 60 ft. and 100 ft. at each bridge pier site.

5.8.6.b Other Recommendations. Indirect recommendations from other sources, such as the local geological information, the "Requirements for Roadway Soil Survey" by Indiana State Highway Commission, as quoted in McKittrick (63), etc., are as follows:

1. The borings are located alternatively right and left on the roadway centerline at 300 ft. spacings. Therefore, 62 borings are required along the highway. The boring depths are 6 ft. or two thirds the height of the fill (whichever is greater). Hand borings and truck mounted borings with split spoon sampling are recommended.
2. At least one boring should encounter rock, cores should be obtained for a depth of 5 to 10 ft. to make sure that sound bedrock has been reached. If there is evidence of solution channels or deep weathering, the cores should be continued into sound rock. The depth of rock core boring is estimated to be 160 ft.
3. Where possible, ground water observation should be made at the time the borings are completed and twenty-four hours afterwards.
4. Routine classification tests, such as grain size distribution and Atterberg limits, should be conducted on samples of each stratum encountered on the project.
5. Consolidation and unconsolidated undrained triaxial tests are performed on Shelby tube samples for the analyses of settlement and bearing capacity of the embankment and other structures.
6. Compaction and CBR tests are performed in cut areas. If the local materials are not used as fill, these tests will not be necessary, but will still be run on borrow materials. Ten cut areas are assumed along the highway.

5.8.6.c Conclusions

1. The data bank can give a general impression of the subsurface materials and permit prediction of efficient drilling and sampling equipment.
2. Using the data bank the number, type and depths of samples can be better estimated.
3. The expected values from testing of extracted samples can be obtained from the data bank and used in quality control for the actual experimental measurements.

5.8.6.d Summary. The quantity estimates of the recommended subsurface investigation program are shown in Table 5-2.

5.9 Example 2

The following example illustrates how state, county, and city engineers may be supplied with presumptive CBR data for preliminary pavement designs through reference to the Indiana data bank.

5.9.a Project Identification

There is to be a new route located in the north of the city of Evansville, Vanderburg County. The city engineer needs CBR data for preliminary pavement design.

5.9.b General Scope of Procedures

The compiled results of the Indiana geotechnical data bank aid in the preliminary phase of pavement design. To accomplish this objective, the investigation is divided into

Table 5-2 The quantity estimates of the recommended subsurface investigation program

Borings and Sampling	
	Qty.
Hand borings	372 Lft
Truck mounted borings with split spoon sampling	1205 Lft
Truck mounted rock core borings	160 Lft
3-in undisturbed samples	15 Ea.
Split spoon samples	44 Ea.
Laboratory testings	
Grain size distribution	62 Ea.
Liquid limit	62 Ea.
Plastic limit	62 Ea.
Natural moisture content	72 Ea.
Consolidation test	3 Ea.
Unconsolidated undrained triaxial test	14 Ea.
pH test	18 Ea.
OMC test*	10 Ea.
Maximum dry density test*	10 Ea.
CBR test*	10 Ea.
Bridge foundation soil analysis and recommendation	6 Ea.

*If excavation materials are not used as fill, these tests will be performed in different quantities.

Table 5-2 (continued)

Embankment settlement analysis	1 Ea.
Stability analysis of retaining walls	5 Ea.
Soils report	LS
Resident soils engineers	Ea.

two parts:

(1) determination of the soil types and their corresponding CBR values within the area;

(2) recommendation of preliminary design CBR values.

5.9.c Determination of the Soil Types and Their Corresponding CBR Values

The soil association was found to be McGary in the Wabash Lowland. From an examination of its statistical soil profile (Table A-III-23, Appendix A) it was found that the texture of the top ten feet was silty clay loam (A-4, A-6, A-7-6). The medians of soaked CBR at 100% maximum dry density (CBR SO1) and soaked CBR at 95% maximum dry density (CBR SO2) are 6.75 and 3.93 respectively. In order to confirm the results, the tables of remolded soil characteristics within the Wabash Lowland (Table A-II-22 to A-II-24) were then examined. It was found that for A-4 soils the medians of CBR SO1 and CBR SO2 were 10.00 and 5.60 respectively; for A-6 soils the medians of CBR SO1 and CBR SO2 were 7.25 and 4.47, respectively; and for A-7-6 soils the medians of CBR SO1 and CBR SO2 were 5.80 and 4.00, respectively.

5.9.d Recommendation of the Preliminary Design CBR Values

With this information it is recommended that the CBR SO1 value be 6.75 and the CBR SO2 value be 3.92 as the medians are used.

5.10 Example 3

It is desired to predict the generalized line of optimums for Standard AASHTO laboratory compaction for the soils of Indiana.

5.10.a Procedures

A scatter plot of ρ_{dmax} vs. OMC was first examined. A strong curvilinear trend of decreasing ρ_{dmax} with increasing OMC was shown. A second degree polynomial model was used for best fit.

5.10.b Results

The equation was found to be

$$\rho_{dmax}(pcf) = 150.667 - 3.016\ OMC + 0.0333\ (OMC)^2,$$

for which $|R| = 0.906$, s.d. of est. = 3.691, and No. of cases = 701. The scatter plot with its best-fit line is shown in Figure 5-5. For example, if OMC = 16.00%, ρ_{dmax} = 110.93 pcf.

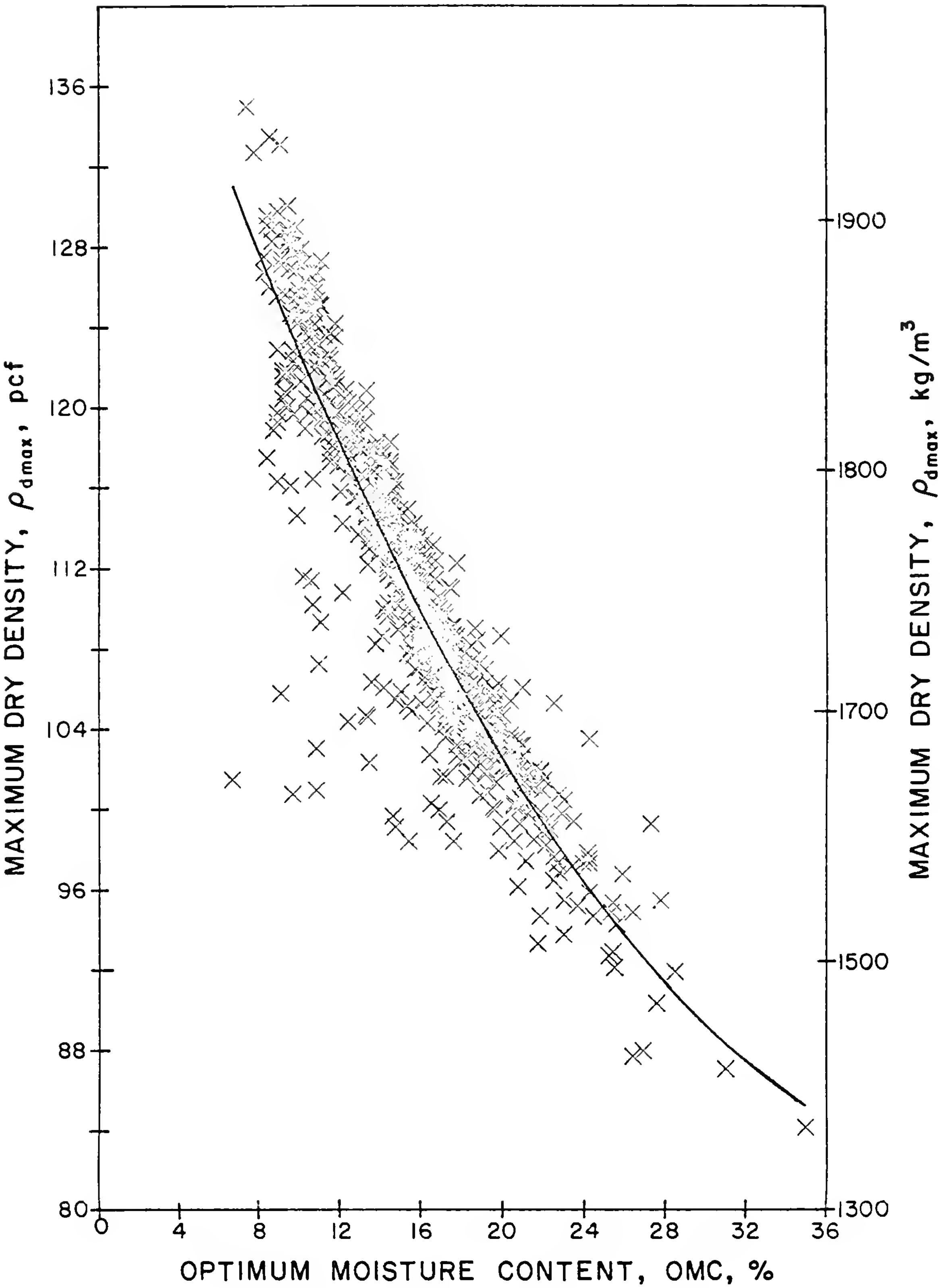


Figure 5-5 Maximum Dry Density vs Optimum Moisture Content, Indiana Soils

CHAPTER 6 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

A computerized, data storage and retrieval system has been developed for the State of Indiana. Both conventional and nonparametric statistical methods have been employed in the analysis of these data. The studies on the topographic characteristics versus physiographic region (Appendix A-1) were based on a one-way classification layout. The results give a general impression of the topographic features of a physiographic region and can be used to make comparisons of overall topographic features between any two such regions. The methods can also be applied to smaller areas if topographic data are adequately distributed. The studies on the remolded soil characteristics versus physiographic regions and AASHTO classifications (Appendix A-II) were based on a two-way classification layout, and show the relationships on a regional basis. The studies on statistical soil profiles (Appendix A-III) were based on a factorial experiment. The results show the general subsoil conditions qualitatively and the estimates of soil characteristics quantitatively with depths for soil associations on a regional basis. Finally, the regression analysis

(Appendix A-IV) shows the functional relationships between design parameters and index properties.

The examples shown in Chapter 5 illustrates specific uses of the geotechnical data bank.

6.2 Conclusions

(a) Topographic features vary with physiographic regions. (Section 4.2.a)

(b) The remolded soil characteristics can be evaluated and contrasted between physiographic regions but also within AASHTO classifications. (Appendix A-II.)

(c) The distributional data confirm that the more plastic* soils have lower maximum dry density and higher optimum moisture content values (Section 4.2.b).

(d) The soil association is the most refined unit for grouping soils to generate soil profiles at the present stage of study (Section 4.2.d).

(e) Statistical regressions show that as the liquid limit or plastic limit of a soil increases, the optimum moisture content increases but maximum dry density decreases. This confirms the findings of earlier authors (Section 4.3.a and Appendix A-IV).

(f) A unique relationship exists between optimum moisture content and maximum dry density which confirms the findings of earlier authors (Section 4.3.a and Appendix A-IV).

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* Soils with higher values of the plasticity index.

(g) The maximum dry density is not significantly influenced by geological factors but the optimum moisture content is (Appendix A-IV).

(h) The CBR value is a function of plasticity characteristics and a correlation between the CBR value at 100% maximum dry density and the CBR value at 95% maximum dry density exists which confirm the findings of earlier authors (Section 4.3.a and Appendix A-IV).

(i) Compression index (C_c) is a function of natural moisture content, initial void ratio, and liquid limit and is significantly influenced by geological factors, which confirms the findings of earlier authors. With 95% confidence the recompression index (C_r) lies in the range of, approximately, $\frac{1}{6.5}\ C_c$ and $\frac{1}{8}\ C_c$ for Indiana soils (Section 4.3.b and Appendix A-IV).

(j) The preconsolidation pressure (p_c) is a function of natural water content, initial void ratio, and natural dry density and is significantly influenced by geological factors. But the scatter in the data is large (Section 4.3.b and Appendix A-IV).

(k) No definite correlation of strength angle and cohesion intercept versus plasticity characteristics was found for Indiana soils (Section 4.3.c).

(l) Non-parametric statistical methods are preferred, as opposed to conventional statistical methods, for data analyses (Section 3.3.1.a).

(m) The data bank is valuable for making recommendations for preliminary design of geotechnical works (Chapter 5).

(n) The physiographic region, engineering soil classification, soil association, and a combination of them were used as grouping units. The interquartile ranges (IR's) as shown in Appendix A-II and A-IV for most soil characteristics are small and tolerable. In other words, a good homogeneity of soil characteristics is evidenced with these groupings.

(o) It is emphasized that the data bank is not proposed as a substitute for fuller site investigation, sampling and testing, but as a framework against which various test results can be judged for their consistency and reliability.

6.3 Recommendations for Future Research

(a) In addition to displaying the distribution of each data item based on physiographic region, or engineering soil classification, or soil association or a combination of them, cluster analysis and principle components analysis (23, 28, 68) are recommended for future research. These combine

the characteristics of each sample into a general geotechnical character for the soil and group the samples into similar geotechnical zones. The proper selection of soil characteristics will play an important role in correctly identifying geotechnical zones.

(b) The feasibility of adding information to the existing data file should be studied.

(c) The feasibility of establishing a computerized geotechnical information library, in which data sources can be searched and attached to the data bank, should be studied. The data sources include more subsurface investigation reports, geological surveys, agricultural soil surveys, and published relationships among soil characteristics in the State of Indiana.

(d) The feasibility of establishing a permanent facility to operate and maintain the data bank and to provide services to potential users, such as private consultants and contractors should be studied. This facility would keep the data file current and would make revisions of existing correlations and regression equations among soil characteristics.

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APPENDICES

The following Appendices are not included in this copy of this Report:

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A copy of any of the above listed Appendices may be obtained for the cost of reproduction by writing to:

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*Pages 459 through 472 of this Appendix are Codes referred to in this Report and are included in this copy of the Report for reference purposes.

APPENDIX A-IV

REGRESSIONS EQUATIONS AND CORRELATIONS

A. Compaction parameters:

1. <u>Maximum dry density ($\rho_{d\ max}$):</u>

$$\hat{\rho}_{d\ max} = 135.843 - 1.279\ w_p \text{ (in pcf)}$$
$$= 2202.563 - 20.743\ w_p \text{ (in kg/m}^3\text{)},$$

for which R = − 0.692, s.d. of est. = 6.434, and n = 601.

$$\hat{\rho}_{d\ max} = 128.338 - 0.463\ w_L \text{ (in pcf)}$$
$$= 2080.870 - 7.510\ w_L \text{ (in kg/m}^3\text{)},$$

for which R = 0.744, s.d. of est. = 5.651, and n = 601.

$$\hat{\rho}_{d\ max} = 142.888 - 0.554\ w_L - 0.727\ w_p + 0.00849\ w_L w_P - 0.0900 \text{ silt (in pcf)}$$
$$= 2316.786 - 8.982\ w_L - 11.787\ w_p + 0.138\ w_L w_P - 1.459 \text{ silt (in kg/m}^3\text{)},$$

for which |R| = 0.808, s.d. of est. = 4.994, and n = 601.

2. <u>Optimum moisture content (OMC):</u>

$$\widehat{OMC} = 4.464 + 0.619\ w_p \text{ (in \%)},$$

for which R = 0.698, s.d. of est. = 2.905, and n = 596.

$$\widehat{OMC} = 7.626 + 0.237\ w_L \text{ (in \%)},$$

for which R = 0.794, s.d. of est. = 2.464, and n = 596.

$$\widehat{OMC} = 7.457 = 0.0369 \text{ sand} + 0.0174 \text{ silt} + 0.171\ w_L + 0.155\ w_P - 0.413\ x_1 - 0.401\ x_2 - 1.740\ x_3 + 0.968\ x_4 - 0.409\ x_5 + 0.503\ x_6 - 0.186\ x_7 - 0.560\ x_8 + 0.336\ x_9 - 0.463\ x_{10} - 0.752\ x_{11} \text{ (in \%)},$$

for which $|R|$ = 0.843, s.d. of est. = 2.210, and n = 596*.

*Note: No data available for physiographic region of Maumee Lacustrine Section (x_{12}).

3. Maximum dry density vs. optimum moisture content:

$$\hat{\rho}_{d\ max} = 150.667 - 3.016\ OMC + 0.0333\ (OMC)^2,$$

for which $|R|$ = 0.906, s.d. of est. = 3.691, n = 701.

4. CBR at 100% maximum dry density (CBR S01):

$$\log \hat{CBR}\ S01 = 1.204 + 0.145 \log w_L - 0.137 \log PI - 0.149 (\log PI)(\log w_L) - 0.0778\ z_1 - 0.109\ z_2 + 0.112\ z_3 - 0.0505\ z_4 - 0.0858\ z_5 - 0.122\ z_6 - 0.0575\ z_7 - 0.0633\ z_8 - 0.0587\ z_9 - 0.105\ z_{10} - 0.688\ z_{11},$$

for which $|R|$ = 0.620, s.d. of est. = 0.166, and n = 493.

5. CBR at 100% maximum dry density (CBR S01) vs. CBR at 95% maximum dry density (CBR S02):

$$\hat{CBR}\ S02 = 1.339 + 0.433\ CBR\ S01,$$

for which R = 0.851, s.d. of est. = 2.010, and n = 553.

$$\hat{CBR}\ S02 = 0.051 + 0.667\ \hat{CBR}\ S01 - 0.00760\ (CBR\ S01)^2,$$

for which $|R|$ = 0.864, s.d. of est. = 1.515, and n = 553.

B. Consolidation parameters:

1. Compression index (C_c):

$$\hat{C}_c = 0.00797\ (w_L - 8.16),$$

for which R = 0.829, s.d. of est. = 0.116, and n = 312.

$$\hat{C}_c = 0.0126\ w_n - 0.162,$$

for which R = 0.925, s.d. of est. = 0.112, and n = 332.

$$\hat{C}_c = 0.496\ e_o - 0.195,$$

for which R = 0.873, s.d. of est. = 0.143, and n = 335.

$$\hat{C}_c = - 0.151 + 0.00326\ w_n + 0.191\ e_o + 0.00325\ w_L + 0.0162\ x_1 - 0.0110\ x_2 + 0.0208\ x_3 + 0.0296\ x_4 + 0.0120\ x_5 - 0.0110\ x_6 + 0.0365\ x_7 + 0.0351\ x_8 + 0.0646\ x_9 + 0.0649\ x_{10} + 0.0594\ x_{11} - 0.0245\ z_1 - 0.0313\ z_2 - 0.00987\ z_3 - 0.0917\ z_4 - 0.121\ z_6 - 0.0292\ z_7 - 0.0667\ z_8 + 0.00841\ z_9 - 0.0418\ z_{10} - 0.00884\ z_{11},$$

for which |R| = 0.952, s.d. of est. = 0.0670, and n = 302*.

*Note: Note data available for physiographic region of Maumee Lacustrine Section (x_{12}) and parent material of soils formed in loamy Wisconsin age glacial till (z_5).

2. Compression ratio (C'_r):

$$\hat{C}'_r = 0.0125 + 0.152\ e_o,$$

for which R = 0.704, s.d. of est. = 0.0448, and n = 333.

$$\hat{C}'_r = 0.0249 + 0.003\ w_n,$$

for which R + 0.701, s.d. of est. + 0.0361, and n = 325.

$$\hat{C}'_r = 0.0294 + 0.00238\ w_L,$$

for which R = 0.665, s.d. of est. = 0.0373, and n = 309.

3. Compression index (C_c) vs. compression ratio (C'_r):

$$C_c = 0.0844 + 9.121\ (C'_r)^2,$$

for which |R| = 0.948, s.d. of est. = 0.0928, and n = 339.

4. Recompression index (C_r) vs. compression index (C_c):

$$C_r = - 0.00327 + 0.139\ C_c,$$

for which R = 0.743, s.d. of est. = 0.0173, and n = 298.

5. <u>Preconsolidation pressure (p_c):</u>

$$LI = 0.770 - 0.115 \log p_c \text{ (Tsf)},$$

for which R = - 0.200, s.d. of est. = 0.554, and n = 311.

$$\begin{aligned}\hat{p}_c &= 107.928 - 144.156 \log w_n + 30.687 (\log w_n)^2 \\ &+ 8.0064 (\log e_o)^2 - 3.448 (\log \rho_d)^3 \\ &+ 22.949 (\log w_n)(\log \rho_d) - 1.163\, x_1 \\ &- 1.336\, x_2 + 0.240\, x_3 - 0.932\, x_4 - 1.297\, x_5 \\ &- 0.738\, x_6 - 0.872\, x_7 - 0.911\, x_8 - 1.286\, x_9 \\ &+ 0.195\, x_{11} \text{ (}P_c\text{ in Tsf and } \rho d \text{ in pcf)} \\ &= 10340.00 - 13810.144 \log w_n + 2939.815 (\log w_n)^2 \\ &+ 767.013 (\log e_o)^2 - 330.318 \left(\log \frac{\rho_d}{16.214}\right)^3 \\ &+ 2198.514 (\log w_n)\left(\log \frac{\rho_d}{16.214}\right) - 111.415\, x_1 \\ &- 127.989\, x_2 + 22.992\, x_3 - 89.286\, x_4 - 124.253\, x_5 \\ &- 70.700\, x_6 - 83.538\, x_7 - 87.274\, x_8 - 123.200\, x_9 \\ &+ 18.681\, x_{11} \text{ (}P_c\text{ in kPa and } \rho_d \text{ in kg/m}^3\text{)},\end{aligned}$$

for which |R| = 0.660, s.d. of est. = 705, and n = 243.*

*Note: No data available for physiographic regions of Valparaiso Moraine and Maumee Lacustrine Section.

C. Strength parameters:

1. <u>Unconfined compressive strength (q_u):</u>

$$\begin{aligned}\log \hat{q}_u &= 0.404 - 0.0205\, w_n \text{ (in Tsf)} \\ &= 2.385 - 0.0205\, w_n \text{ (in kPa)},\end{aligned}$$

for which R = - 0.500, s.d. of est. = 0.254, and n = 1030.

$$\begin{aligned}\log \hat{q}_u &= - 1.554 + 0.0144\, \rho_d \text{ (}q_u\text{ in Tsf and } \rho_d \text{ in pcf)} \\ &= 0.427 + 8.881 \times 10^{-4}\, \rho_d \text{ (}q_u\text{ in kPa and } \rho_d \text{ in kg/m}^3\text{)},\end{aligned}$$

for which R = 0.522, s.d. of est. = 0.250, and n = 1030.

$$\log \hat{q}_u = -0.0425 - 0.264 \text{ LI (in Tsf)}$$
$$= 1.940 - 0.264 \text{ LI (in kPa)},$$

for which R = - 0.366, s.d. of est. = 0.267, and n = 992.

$$\begin{aligned}\log \hat{q}_u = & -1.101 - 0.00740\, w_n + 0.0112\, \rho_d \\ & + 0.0000319\, w_n\rho_d - 0.0883\, x_1 - 0.0518\, x_2 \\ & - 0.0224\, x_3 - 0.0553\, x_4 - 0.0568\, x_5 \\ & - 0.0440\, x_6 - 0.0956\, x_7 - 0.0346\, x_8 \\ & - 0.0140\, x_9 + 0.0724\, x_{11} + 0.172\, z_1 \\ & + 0.0568\, z_2 + 0.109\, z_3 + 0.0612\, z_4 \\ & - 0.163\, z_5 - 0.165\, z_6 + 0.0320\, z_7 \\ & + 0.173\, z_8 - 0.0133\, z_9 + 0.00343\, z_{10} \\ & + 0.0279\, z_{11} \text{ } (q_u \text{ in Tsf and } \rho_d \text{ in pcf}) \\ = & \ 0.880 - 0.00740\, w_n + 6.910 \times 10^{-4}\, \rho_d \\ & + 1.967 \times 10^{-6}\, w_n\rho_d - 0.0883\, x_1 \\ & - 0.0518\, x_2 - 0.0224\, x_3 - 0.553\, x_4 \\ & - 0.0568\, x_5 - 0.0440\, x_6 - 0.0956\, x_7 \\ & - 0.0346\, x_8 - 0.0140\, x_9 + 0.0724\, x_{11} \\ & + 0.0568\, z_2 + 0.109\, z_3 + 0.0612\, z_4 \\ & - 0.163\, z_5 - 0.165\, z_6 + 0.0320\, z_7 \\ & + 0.173\, z_8 - 0.0133\, z_9 + 0.00343\, z_{10} \\ & + 0.0279\, z_{11} \text{ } (q_u \text{ in kPa and } \rho_d \text{ in kg/m}^3),\end{aligned}$$

for which |R| = 0.570, s.d. of est. = 0.244, and n = 1030.*

*Note: No data available on physiographic regions of Valparaiso Moraine (x_{10}) and Maumee Lacustrine Section (x_{12}).

2. Liquid limit vs. SPT

$$
\begin{aligned}
w_L = {} & 47.946 + 0.495\ w_n - 9.931 \log SPT \\
& + 0.488\ w_n \log SPT - 19.323\ x_1 - 25.121\ x_2 \\
& - 30.455\ x_3 - 17.775\ x_4 - 28.875\ x_5 \\
& - 24.529\ x_6 - 26.068\ x_7 - 26.272\ x_8 \\
& - 24.100\ x_9 - 16.493\ x_{10} - 37.182\ x_{11} \\
& + 9.104\ z_1 - 2.930\ z_2 - 3.809\ z_3 - 0.378\ z_4 \\
& - 12.178\ z_5 - 5.404\ z_6 - 7.021\ z_7 + 23.069\ z_8 \\
& - 0.249\ z_9 + 0.962\ z_{10} + 13.878\ z_{11} \text{ (in \%)},
\end{aligned}
$$

for which $|R| = 0.859$, s.d. of est. = 19.503, n = 533.

3. Unconfined compressive strength (q_u) vs. SPT

$$
\begin{aligned}
q_u &= 0.577 + 0.0265 \text{ SPT (in Tsf)} \\
&= 55.277 + 2.539 \text{ SPT (in kPa)},
\end{aligned}
$$

for which $|R| = 0.400$, s.d. of est. = 0.567, and n = 86.

4. Strength angle (ϕ) vs. SPT

$$\phi = 26.052 - 0.743 \text{ SPT (in degree)}$$

for which $|R| = - 0.450$, s.d. of est. = 5.357, and n = 24.

APPENDIX B: CODING SYSTEMS AND COMPUTER PROGRAMS

APPENDIX B-1

ERRATA TO THE USER'S MANUAL OF "THE DEVELOPMENT OF THE COMPUTERIZED GEO-TECHNICAL DATA BANK FOR THE STATE OF INDIANA" BY GARY D. GOLDBERG (35)

(a) The following coding system for soil association (ASSOC) is to replace the system as described in pp. 127-129, Goldberg (35).

VARIABLE NAME: ASSOC

CODE	DESCRIPTION
1.	Eel-Martinsville-Gemesee
2.	Genesee-Ross-Shoals
3.	Wakeland-Stendal-Haymond-Bartle
4.	Genesee-Shoals-Eel
5.	Haymond-Nolin-Petrolia
6.	Genesee-Eel-Stendal-Pope
7.	Huntington-Wheeling-Markland
8.	Huntington-Lindside
9.	Haymond-Wakeland
10.	Alida-Del Rey-Whitaker
11.	Bono-Maumee-Warners
12.	Chelsea-Hillsdale-Oshtemo
13.	Conrad-Wooten-Weiss
14.	Door-Tracy-Quinn
15.	Door-Lydick
16.	Elston-Wea
17.	Dubois-Otwell-Bartle
18.	Fox-Martinsville-Aluvi
19.	Fox-Nineveh-Ockley
20.	Fox-Rodman
21.	Fulton-Rimer-Milford-Rensselaer
22.	Homer-Sebewa-Gilford
23.	Maumee-Gilford-Rensselaer
24.	Maumee-Newton
26.	Martinsville-Bellmore-Fox
27.	Martinsville-Whitaker
28.	Mahalasville-Whitaker
30.	Milford-Montgomery-Rensselaer

VARIABLE NAME	ASSOC (continued)
CODE	DESCRIPTION
31.	McGary
32.	Negley-Parke
33.	Oshtemo-Bronson
34.	Oakville-Plainfield-Adrian
35.	Oshtemo-Fox
36.	Ockley-Westland
37.	Ockley-Wea
38.	Ockley-Fox
39.	Plainfield-Brems-Morrocco
40.	Plainfield-Tyner-Oshtemo
41.	Plainfield- Watseka
42.	Plainfield-Chelsea
43.	Patton-Henshaw
44.	Patton-Lyles-Henshaw
45.	Peoga-Bartle-Hosmer
46.	Parke-Miami-Negley
47.	Rensselaer-Montgomery
48.	Rensselaer-Darroch
49.	Rensselaer-Whitaker
50.	Vincennes-Zipp-Ross
51.	Volinia
53.	Wea-Crane
54.	Warsaw-Elston-Fox
55.	Westland-Sleeth
56.	Weinbach-Sciotoville
57.	Weinbach-Wheeling
58.	Crosier-Brookston
59.	Brookston-Odell-Corwin
61.	Blount-Morley-Pewamo
62.	Blount-Pewamo
63.	Riddles-Miami-Crosier

VARIABLE NAME	ASSOC (continued)
CODE	DESCRIPTION
64.	Crosby-Brookston
65.	Elliott-Markham-Pewamo
66.	Fincastle-Ragsdale-Brookston
67.	Hoytville-Nappanee
69.	Parr-Miami
70.	Parr-Corwin
71.	Randolph-Millsdale
72.	Reesville-Ragsdale
73.	Raub-Ragsdale
74.	Ragsdale-Sidell
76.	Russell-Hennepin
77.	Russell-Xenia
78.	Miami-Metea-Celina
79.	Miami-Owosso-Riddles
80.	Miami-Crosier-Metea
81.	Miami-Russell-Fincastle
82.	Miami-Fox-Milton
83.	Miami-Crosby
84.	Miami-Hennepin
85.	Miami-Fox-Martinsville
86.	Morley-Blount
87.	Muskingum-Shadeland-High Gap
88.	Odell-Chalmers
89.	Sidell-Parr
90.	Hennepin-Rodman
91.	Avonburg-Clermont
92.	Cincinnati-Hickory
93.	Cincinnati-Rossmoyne-Hickory
95.	Cincinnati-Ava
96.	Cincinnati-Ava-Alford
98.	Crider-Hagerstown-Frederick
99.	Crider-Frederick
100.	Corydon-Weikert-Berks

VARIABLE NAME	ASSOC (continued)
CODE	DESCRIPTION
101.	Fairmount-Switzerland
102.	Grayford
103.	Lawrence-Bedford-Crider
104.	Tilsit-Johnsburg
105.	Wellston-Zanesville-Berks
106.	Berks-Gilpin-Weikert
107.	Zanesville-Wellston
108.	Mucks-Peats
109.	Alford
110.	Bloomfield-Princeton-Ayrshire
111.	Hosmer
112.	Iva-Ava
113.	Hosmer-Cincinnati-Iva
114.	Lyles-Ayrshire-Princeton
115.	Princeton-Ayrshire-Bloomfield
116.	Princeton-Fox

(b) The following listing of computer program is to replace the Program Number 3 as described in pp. 152-160, Goldberg (35)

```
FILE NAME       PROG
RUN NAME        **********  SOIL DATA FOR THE STATE OF INDIANA  **********
VARIABLE LIST   COUNTY,HOLENO,SAMPNO,DISTRI,DATEYR,DATEMO,DATEDA,TOWN,TOWNDI,
                RANGE,RANGDI,SECTIO,PROJPR,PROJNO,PROJPA,PROJMI,CONTPR,
                CONTNO,ROADPR,ROADNO,ROADSU,BORING,ASSOC,REPEAT,STATNO,OFFSET,
                OFFDIR,LINE1,LINE2,SOURCE,SAMPTY,LABNO,GRDSUR,DEPTHT,
                DEPTHB,SPT,PHYSIO,SERIES,PARENT,HORIZO,SLOPE,EROSIO,BEDRKS,
                BEDRKB,WATERS,WATERC,WATERF,DRAIN,PERMEA,FLOOD,FROST,SHRINK,PH,
                GRAD01 TO GRAD10,SAND,SILT,CLAY,COLL,LL,PL,PI,SL,LOSSIG,
                NATMC,NATWD,NATDD,SPECGR,TEXTUR,ORGANI,COLOR,TESTEF,MAXDD,
                MAXWD,OPTIMC,CBRUN1,CBRUN2,CBRSO1,CBRSO2,QUSTR,AASHTO1,UNIF1,
                QUSTA,TYPE,STRENGTH,STRAIN,CONFPRES,COHESION,ANGLE,POREPRES,
                MAJOR,EO,EF,SO,SF,PO,PC,CC,CR,CV
INPUT MEDIUM    SOIL1
INPUT FORMAT    (1X,F2.0,F5.0,F2.0,1X,F1.0,1X,F2.0,1X,F2.0, 1X,F2.0, 1X,F2.0,
                1X,F1.0,1X,F2.0, 1X,F1.0,1X,F2.0,1X,A3,  A5,A3,F3.0,1X,A3,
                F5.0, 1X,A2,F3.0,A1, 1X,A8, 1X,F3.0, 1X,F1.0/
                11X,F7.0, 1X,F4.0, 1X,F1.0, 1X, A8,A2, 1X,F2.0, 1X,F2.0, 1X, A8,
                1X,F5.1,   1X,F4.1, 1X,F4.1, 1X,F2.0, 1X,F2.0, 1X,F4.0/
                11X,F2.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F3.1, 1X,F4.1, 1X,F2.1,
                1X,F4.1, 1X,F4.1, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0,
                1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1/ 11X,
                F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1,
                1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1/
                11X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.3, 1X,F2.0, 1X,F1.0, 1X,F2.0,
                1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1, 1X,F3.1, 1X,F3.1,
                1X,F3.1, 1X,F4.2,F2.0,F2.0/ 11X F4.2,1X,F1.0,3F4.2,F3.2,
                F3.1,2F4.2,1X,2F4.3,2F4.1,2F4.2,2F4.3,F4.2)
VAR LABELS      COUNTY COUNTY/HOLENO HOLE NUMBER/SAMPNO SAMPLE NUMBER/DATEYR YEAR
                TAKEN FROM HOLE/DATEMO MONTH TAKEN FROM HOLE/DATEDA DAY TAKEN FR
                OM HOLE/TOWN TOWNSHIP/TOWNDI TOWNSHIP-DIRECTION/RANGE RANGE/
                RANGDI RANGE-DIRECTION/SECTIO SECTION/
                PROJPR PROJECT NUMBER-PREFIX/PROJNO PROJECT NUMBER/
                PROJPA PROJECT NUMBER-PARENTHESIS/PROJMI PROJECT NUMBER-MILE/
                CONTPR CONTRACT NUMBER-PREFIX/
                CONTNO CONTRACT NUMBER/  DISTRI DISTRICT/
                ROADPR ROAD NUMBER-PREFIX/ROADNO ROAD NUMBER/
                ROADSU ROAD NUMBER-SUFFIX/BORING BORING NUMBER/ASSOC SOIL ASSOCIA
                TION/REPEAT DATA REPEAT/
                STATNO STATION NUMBER/OFFSET OFFSET/OFFDIR OFFSET-DIRECTION/
                LINE1 TO LINE2=LINE NUMBER/SOURCE SOURCE OF INFORMATION/
                SAMPTY SAMPLE TYPE/LABNO LAB NUMBER/
                GRDSUR GROUND SURFACE ELEVATION/DEPTHT DEPTH TO TOP OF SAMPLE/
                DEPTHB DEPTH TO BOTTOM OF SAMPLE/PHYSIO PHYSIOGRAPHIC UNIT/
                SPT N VALUE OF SPT/
                SERIES SOIL SERIES NAME/   PARENT PARENT MATERIAL/
                HORIZO HORIZON/   SLOPE SLOPE CLASS/   EROSIO EROSION CLASS/
                BEDRKS DEPTH TO BEDROCK-SOIL SURVEY/   BEDRKB DEPTH TO BEDROCK-BO
                RING LOG/WATERS DEPTH-SEASONAL HIGH WATER TABLE-SOIL SURVEY/
                WATERC WATER DEPTH AT COMPLETION/WATERF WATER DEPTH FINAL OR 24 H
                OURS/DRAIN NATURAL SOIL DRAINAGE/PERMEA PERMEABILITY/
                FLOOD FLOODING POTENTIAL/  FROST POTENTIAL FROST ACTION/
                SHRINK SHRINK-SWELL POTENTIAL/  PH REACTION-PH/
                GRAD01 PERCENT PASSING 1 1-2≠≠ SIEVE/
                GRAD02 PERCENT PASSING 1≠≠ SIEVE/
```

```
               GRAD03 PERCENT PASSING 3-4≠≠ SIEVE/
               GRAD04 PERCENT PASSING 1-2≠≠ SIEVE/
               GRAD05 PERCENT PASSING 3-8≠≠ SIEVE/
               GRAD06 PERCENT PASSING NO. 4 SIEVE/
               GRAD07 PERCENT PASSING NO. 10 SIEVE/
               GRAD08 PERCENT PASSING NO. 40 SIEVE/
               GRAD09 PERCENT PASSING NO. 200 SIEVE/
               GRAD10 PERCENT PASSING NO. 270 SIEVE/
               SAND PERCENT SAND/
               SILT PERCENT SILT/CLAY PERCENT CLAY/COLL PERCENT COLLOIDS/
               LL LIQUID LIMIT/    PL PLASTIC LIMIT/   PI PLASTICITY INDEX/
               SL SHRINKAGE LIMIT/    LOSSIG LOSS ON IGNITION/
               NATMC NATURAL MOISTURE CONTENT/   NATWD NATURAL WET DENSITY/
               NATDD NATURAL DRY DENSITY/   SPECGR SPECIFIC GRAVITY/
               TEXTUR TEXTURAL CLASSIFICATION/   ORGANI ORGANIC CONTENT/
               COLOR COLOR/
               TESTEF TEST-EFFORT IDENTIFIER/    MAXDD MAXIMUM DRY DENSITY/
               MAXWD MAXIMUM WET DENSITY/    OPTIMC OPTIMUM MOISTURE CONTENT/
               CBRUN1 UNSOAKED CBR-100 MAXDD/  CBRUN2 UNSOAKED CBR-95 MAXDD/
               CBRSO1 SOAKED CBR-100 MAXDD/CBRSO2 SOAKED CBR-95 MAXDD/
               QUSTR UNCONFINED COMPRESSIVE STRENGTH-TSF/
               QUSTA FAILURE STRAIN-PERCENT/
               TYPE TYPE OF STRENGTH TEST/STRENGTH FAILURE STRENGTH/
               STRAIN FAILURE STRAIN/CONFPRES CONFINING PRESSURE/
               ANGLE FAILURE ANGLE/POREPRES PORE PRESSURE AT FAILURE/MAJOR MAJOR
               PRINCIPAL STRESS/EO INITIAL VOID RATIO/EF FINAL VOID RATIO/SO IN
               ITIAL DEGREE OF SATURATION/SF FINAL DEGREE OF SATURATION/PO OVERB
               URDEN STRESS/PC PRECONSOLIDATION PRESSURE/CC COMPRESSION INDEX/
               CR RECOMPRESSION INDEX/CV COEFFICIENT OF CONSOLIDATION
RECODE         AASHTO1,UNIF1 (BLANK=9999)/ SAND (BLANK=999)
VALUE LABELS   COUNTY (-0)UNKNOWN (01)ADAMS (02)ALLEN (03)BARTHOLOMEW (4)BENTON
               (05)BLACKFORD (06)BOONE (07)BROWN (08)CARROLL (09)CASS (10)CLARK
               (11)CLAY (12)CLINTON (13)CRAWFORD (14)DAVIESS (15)DEARBORN
               (16)DECATUR (17)DEKALB (18)DELAWARE (19)DUBOIS (20)ELKHART
               (21)FAYETTE (22)FLOYD (23)FOUNTAIN (24)FRANKLIN (25)FULTON
               (26)GIBSON (27)GRANT (28)GREENE (29)HAMILTON (30)HANCOCK
               (31)HARRISON (32)HENDRICKS (33)HENRY (34)HOWARD (35)HUNTINGTON
               (36)JACKSON (37)JASPER (38)JAY (39)JEFFERSON (40)JENNINGS
               (41)JOHNSON (42)KNOX (43)KOSCIUSKO (44)LAGRANGE (45)LAKE
               (46)LAPORTE (47)LAWRENCE (48)MADISON (49)MARION (50)MARSHALL
               (51)MARTIN (52)MIAMI (53)MONROE (54)MONTGOMERY (55)MORGAN
               (56)NEWTON (57)NOBLE (58)OHIO (59)ORANGE (60)OWEN (61)PARKE
               (62)PERRY (63)PIKE (64)PORTER (65)POSEY (66)PULASKI (67)PUTNAM
               (68)RANDOLPH (69)RIPLEY (70)RUSH (71)ST. JOSEPH (72)SCOTT
               (73)SHELBY (74)SPENCER (75)STARKE (76)STEUBEN (77)SULLIVAN
               (78)SWITZERLAND (79)TIPPECANOE (80)TIPTON (81)UNION
               (82)VANDERBURGH (83)VERMILLION (84)VIGO (85)WABASH (86)WARREN
               (87)WARRICK (88)WASHINGTON (89)WAYNE (90)WELLS (91)WHITE
               (92)WHITLEY (93)STATE OF KENTUCKY/
               DISTRI (-0)UNKNOWN (1)CRAWFORDSVILLE
               (2)FORT WAYNE (3)GREENFIELD (4)LAPORTE (5)SEYMOUR (6)VINCENNES/
               DATEMO (-0)UNKNOWN (01)JANUARY (02)FEBRUARY (03)MARCH (04)APRIL
               (05)MAY (06)JUNE (07)JULY (08)AUGUST (09)SEPTEMBER (10)OCTOBER
               (11)NOVEMBER (12)DECEMBER/
               TOWNDI (-0)UNKNOWN (1)NORTH (2)SOUTH/
```

RANGDI (-0)UNKNOWN (1)EAST (2)WEST/
ASSOC (-0)UNKNOWN
(1) EEL-MARTINSV-GENESSE (2) GENESEE-ROSS-SHOALS (3) WAKELAN-STEN
DA-HAYMO (4) GENESEE-SHOALS-EEL (5) HAYMON-NOLIN-PETROLI
(6) GENESE-EEL-STEND-POPE
(7) HUNTING-WHEELI-MARKH (8) HUNTINGTON-LINDSIDE
(9) HAYMOND-WAKELAND (10) ALIDA-DELREY-WHITAKE (11) BONO-MAUMEE
-WARNERS (12) CHELS-HILLSDAL-OSHTE (13) CONRAD-WOOTEN-WEISS (14)
DOOR-TRACY-QUINN (15) DOOR-LYDICK (16) ELSTON-WEA (17) DUBOIS-OTW
ELL (18) FOX-MARTINSVILL-ALUV (19) FOX-NINEVEH-OCKLEY (20) FOX-RO
DMAN (21) FULT-RIM-MILF-RENSSEL (22) HOMER-SEBEWA-GILFORD (23) MA
UME-GILFOR-RENSSEL (24) MAUMEE-NEWTON (26) MARTINSVIL-BELMO-FOX
(27) MARTINSVILLE-WHITAKE (28) MAHALASVILL-WHITAKER (30)MILF-MONT
GOM-RENSSEL (31) MCGARY (32) NEGLEY-PARKE (33) OSHTEMO-BRONSON
(34) OAKVIL-PLAINFIE-TAWA (35) OSHTEMO-FOX (36) OCKLEY-WESTLAND
(37) OCKLEY-WEA (38) OCKLEY-FOX (39) PAILNFIE-BREM-MORROC (40) P
LAINFIE-TYNER-OSHTE (41) PLAINFIELD-WATSEKA (42) PLAINFIELD-CHELS
EA (43) PATTON-HENSHAW (44) PATTON-LYLES-HENSHAW (45) PEOGA-BARTL
E-HOSMER (46) PARKE-MIAMI-NEGLEY (47) RENSSELAER-MONTGOMER (48)
RENSSELAER-DARROCH (49) RENSSELAER-WHITAKER (50) VINCENNES-ZIPP-R
OSS (51) VOLINIA-DICKINSON (53) WEA-CRANE (54) WARSAW-ELSTON-FOX
(55) WESTLAND-SLEETH (56) WEINBACH-SCIOTOVILLE (57) WEINBACH-WHEE
LING (58) CROSIER-BROOKSTON (59) BROOKSTO-ODELL-CORWI (61) BLOUNT
-MORLEY-PEWAMO (62) BLOUNT-PEWAMO (63) RIDDLES-MIAMI-CROSBY (64)
CROSBY-BROOKSTON (65) ELLIOT-MARKHAM-PEWAM (66) FINCASTLE-RAGSDAL
E (67) HOYTSVILLE-NAPPANEE (69) PARR-MIAMI (70) PARR-CORWIN (71)
RANDOLPH-HILLSDALE (72) REESVILLE-RAGSDALE (73) RAUB-RAGSDALE
(74) RAGSDALE-SIDELL (76) RUSSELL-HENNEPIN (77) RUSSELL-XENIA
(78) MIAMI-METEA-CELINA (79) MIAMI-OSOSSO-RIDDLES (80) MIAMI-C
ROSBY-METEA (81) MIA-RUSSEL-FINCASTLE (82) MIAMI-FOX-MILTON (83)
MIAMI-COSBY (84) MIAMI-HENNEPIN (85) MIAMI-FOX-MARTINSVIL (86) MO
RLEY-BLOUNT (87) MUSKIN-SHADELA-HIGHG (88) ODELL-CHALMERS (89) SI
DELL-PARR (90) HENNEPIN-RODMAN (91) AVONBURG-CLERMONT (92) CINCIN
NATI-HICKORY (93) CINCINNA-ROSSMO-HICK (95) CINCINNATI-AVA (96) C
INCINNAT-AVA-ALFORD (98) CRID-HAGERSTO-FREDER (99) CRIDER-FREDERI
CK (100) CORYDON-WEIKERT-BERK (101) FAIRMOUNT-SWITZERLAN (102) GR
AYFORD (103) LAWRENC-BEDFORD-CRID (104) TILSIT-JOHNSBURG (105) WE
LLST-ZANESVIL-BERK (106) BERKS-GILPIN-WEIKERT (107) ZANESVILLE-WE
LLSTON (108) MUCKS-PEATS (109) ALFORD (110) BLOOMFI-PRINCE-AYRSH
(111) HOSMER (112) IVA-AVA (113) HOSMER-CINCINNAT-IVA (114) LYLE
-AYRSHIR-PRINCET (115) PRINCET-AYRSH-BLOOMF (116) PRINCETON-FOX/
REPEAT (1)ORIGINAL DATA (2)DATA FROM SAME HOLE
(3)DATA-DIFFERENT HOLE/
OFFDIR (-0)UNKNOWN (1)LEFT (2)RIGHT (3)CENTERLINE/
SOURCE (-0)UNKNOWN (01)STATE (02)ATEC-OEA
(03)ATEC-CONSULTANT (04)NUTTING-OEA (05)NUTTING-CONSULTANT
(06)PITTSBURG TEST-OEA (07)PITTSBURG TEST-CONS
(08)WESTENHOFF-NOV-OEA (09)WESTENHOFF-NOV-CONS (10)STOKLEY-OEA
(11)STOKLEY-CONSULTANT (12)STS-OEA (13)STS-CONSULTANT
(14)GEO SURVEYS-OEA (15)GEO SURVEYS-CONSULT. (16)TESTING SERVICE-
OEA (17)TESTING SERVICE-CONSULT. (18)HURST-ROSHE-OEA (19)HURST-RO
SHE-CONSULT. (20)CHASTAIN-OEA (21)CHASTAIN-CONSULTANT
(22)GREGG-OEA (23)GREGG-CONSULTANT (24)SHAFER-OEA (25)SHAFER-CONS
ULTANT/
SAMPTY (-0)UNKNOWN (01)SHELBY TUBE (02)SPLIT SPOON

(03)DENISON SAMPLER (04)CONT. FLIGHT AUGER (05)HAND AUGER
(06)JAR (07)BAG (08)ROCK CORE (09)PISTON SAMPLER (10)HOLLOW STEM
AUGER (11)POWER AUGER-MACHINE/
PHYSIO (-0)UNKNOWN (1)TIPTON TILL PLAIN (2)DEARBORN UPLAND
(3)MUSCATATUCK REGIONAL SLOPE (4)SCOTTSBURG LOWLAND
(5)NORMAN UPLAND (6)MITCHELL PLAIN (7)CRAWFORD UPLAND
(8)WABASH LOWLAND (9)CALUMET LACUSTRINE (10)VALPARAISO MORAINE
(11)KANKAKEE LACUSTRINE (12)MAUMEE LACUSTRINE (13)STEUBEN MORAINA
L/
SERIES (-0)UNKNOWN (10)ADE (20)ADRIAN (30)ALFORD (40)ALGIERS
(50)ALIDA (60)ALLISON (70)ARMIESBURG (80)AUBBEENAUBBEE
(90)AVA (100)AVONBURG (110)AYR (120)AYRSHIRE (130)BARTLE
(140)BAXTER (150)BEDFORD (160)BELLMORE (170)BERKS
(180)BIRDS (190)BLOOMFIELD (200)BLOUNT (210)BONNIE
(220)BONO (230)BOONESBORO (240)BOYER (250)BRADY
(260)BREMS (270)BRONSON (280)BROOKSTON (290)BURGIN
(300)BURNSIDE (310)CAMDEN (320)CARLISLE (330)CASCO
(340)CATLIN (350)CELINA (360)CHALMERS (370)CHELSEA
(380)CINCINNATI (390)CLARENCE (400)CLERMONT (410)COLYER
(420)CONOVER (430)CONRAD (440)CORWIN (450)CORY (460)CORYD
ON (470)COUPEE (480)CRANE (490)CRIDER (500)CROSBY
(510)CROSIER (520)CUBA (530)DANA (540)DARROCH (550)DEL RAY
(560)DICKINSON (570)DOOR (580)DOWAGIAC (590)DUBOIS (600)D
UNNING (610)EDEN (620)EDENTON (630)EDWARDS (640)EEL
(650)ELKINSONVILLE (660)ELLIOTT (670)ELSTON (680)EVANSVILLE
(690)FABIUS (700)FAIRMOUNT (710)FINCASTLE (720)FLANAGAN
(730)FORESMAN (740)FOX-SILT LOAM (741)FOX-LOAM (742)FOX-URBAN LAN
D (750)FREDERICK (760)FULTON (770)GENESEE-SILT LOAM (771)GENESEE-
URBAN LAND (780)GILFORD (790)GILPIN (800)GINAT (810)GLENHALL
(820)GRANBY (830)GRAYFORD (840)GUTHRIE (850) HAGERSTOWN
(860)HANNA (870)HASKINS (880)HAUBSTADT (890)HAYMOND (900)
HENNEPIN (910)HENSHAW (920)HICKORY (930)HIGH GAP (940)HIL
LSDALE (950)HOMER (960)HOOPESTON (970)HOSMER (980)HOUGHTO
N (990)HOYTVILLE (1000)HUNTINGTON (1010)HUNTSVILLE (1020)
IONA (1030)IPAVA (1040)IVA (1050)JASPER (1060)JENNINGS
(1070)JOHNSBURG (1080)JULES (1090)KALAMAZOO (1100)KERSTON
(1110)KINGS (1120)KOKOMO (1130)LANDES (1140)LAWRENCE
(1150)LENAWEE (1160)LINDSIDE (1170)LINKVILLE (1180)LONGLOIS
(1190)LORENZO (1200)LOWELL (1210)LUCAS (1220)LYDICK (1230)
LYLES (1240)MAHALASVILLE (1250)MARKHAM (1260)MARKLAND
(1270)MARTINSVILLE (1280)MARTISCO (1290)MASSIE (1300)MATHER
TON (1310)MAUMEE (1320)MCGARY (1330)MEDWAY (1340)MELLOTT
(1350)MERMILL (1360)METAMORA (1370)METEA (1380)MIAMI-SILT L
OAM (1381)MIAMI-URBAN LAND
(1390)MILFORD (1400)MILLSDALE (1410)MILTON (1420)MONITOR
(1430)MONTGOMERY (1440)MONTMORENCI (1450)MORLEY (1460)MOROC
CO (1470)MUREN (1480)MUSKINGUM (1490)MUSSEY (1500)NAPPANE
E (1510)NEGLEY (1520)NEWARK (1530)NEWTON (1540)NICHOLSON
(1550)NINEVEH (1560)NOLIN (1570)OAKVILLE (1580)OCKLEY
(1590)OCTAGON (1600)ODELL (1610)OSHTEMO (1620)OTWELL
(1630)OWOSSO (1640)PALMS (1650)PARKE (1660)PARR (1670)PATT
ON (1680)PEKIN (1690)PEOGA (1700)PETROLIA (1710)PEWAMO
(1720)PIKE (1730)PINHOOK (1740)PLAINFIELD (1750)PLANO
(1760)POPE (1770)PRINCETON (1780)PROCTOR (1790)QUINN
(1800)RAGSDALE (1810)RAHM (1820)RANDOLPH (1830)RARDEN

(1840)RAUB (1850)RAWSON (1860)REESVILLE (1870)RENSSELAER
(1880)RIDDLES (1890)RIMER (1900)ROBINSON (1910)ROCKCASTLE
(1920)RODMAN (1930)ROSS (1940)ROSSMOYNE (1950)RUNNYMEDE
(1960)RUSH (1970)RUSSELL (1980)ST. CLAIR (1990)SARANAC
(2000)SAUGATUCK (2010)SCIOTOVILLE (2020)SEBEWA (2030)SEWARD
(2040)SHADELAND (2050)SHIPSHE (2060)SHOALS (2070)SIDELL
(2080)SLEETH (2090)SLOAN (2100)SPARTA (2110)STARKS
(2120)STEFF (2130)STENDAL (2140)STONELICK (2150)STOY
(2160)STROLE (2170)SUNBURY (2180)SWITZERLAND (2190)SWYGERT
(2200)SYLVAN (2210)TAGGART (2220)TAMA (2230)TEDROW (2240)
TILSIT (2250)TIPPECANOE (2260)TOLEDO (2270)TORONTO (2280)
TRACY (2290)TRAPPIST (2300)TROXEL (2310)TYNER (2320)UNION
TOWN (2330)VIGO (2340)VINCENNES (2350)VOLINIA (2360)WAKEL
AND (2370)WALLKILL (2380)WARNERS (2390)WARSAW (2400)WASEP
I (2410)WASHTENAW (2420)WATSEKA (2430)WAUSEON (2440)WEA
(2450)WEIKERT (2460)WEINBACH (2470)WELLSTON (2480)WESTLAND
(2490)WHEELING (2500)WHITAKER (2510)WHITSON (2520)WILBUR
(2530)WILLETTE (2540)WINGATE (2550)WOODMERE (2560)WOOLPER
(2570)WYNN (2580)XENIA (2590)ZANESVILLE (2600)ZIPP
(2700)BORROW PIT (2800)URBAN LAND (2900)ALLUVIAL LAND
(3000)GRAVEL PIT (3100)GULLIED LAND(3200)STRIP MINE
(3300)CUT AND FILL (3400)MADE LAND/
PARENT (-0)UNKNOWN (1)LACUSTRINE DEPOSITS (2)OUTWASH AND ALLUVIAL
DEPOSITS (3)EOLIAN SAND DEPOSITS (4)THICK LOESS DEPOSITS (5)LOAMY
WISCONSIN TILL (6)CLAYEY WIS. TILL (7)THIN LOESS-LOAMY WIS TILL
(8)THICK LOESS-LOAM WIS TILL (9)THICK LOESS-ILL TILL
(10)RESIDUUM-SI.ST SH SS (11)RESIDUUM-LS BEDROCK
(12)RES-CALCAREOUS SH LS/
HORIZO (-0)UNKNOWN (1)A HORIZON (2)B HORIZON (3)C HORIZON
(4)D HORIZON/
SLOPE (-0)UNKNOWN (1)LEVEL-NEARLY LEVEL (2)0-2 (3)2-6 (4)6-12
(5)12-18 (6) 18-25 (7)25-35 (8)35+/
EROSIO (-0)UNKNOWN (1)NONE-SLIGHT (2)MODERATE-ERODED (3)SEVERE/
WATERC (-0)UNKNOWN (-1)DRY/
WATERF (-0)UNKNOWN (-1)DRY/
DRAIN (-0)UNKNOWN (1)WELL-EXCESSIVE (2)MODERATELY WELL
(3)SOMEWHAT POORLY (4)POORLY-VERY POORLY/
PERMEA (-0)UNKNOWN (1)LESS THAN .06 (2).06-.2 (3).2-.63 (4).63-2.
0 (5)2.0-6.0 (6)6.0-20 (7)GREATER THAN 20/
FLOOD (-0)UNKNOWN (1)NONE (2)PERCHED-PONDED-HAZARD/
FROST (-0)UNKNOWN (1)VERY LOW (2)VERY LOW-LOW (3)LOW (4)LOW-MODE
RATE (5)MODERATE (6)MODERATE-HIGH (7)HIGH (8)HIGH-VERY HIGH
(9)VERY HIGH/
SHRINK (-0)UNKNOWN (1)VERY LOW (2)VERY LOW-LOW (3)LOW (4)LOW-MODE
RATE (5)MODERATE (6)MODERATE-HIGH (7)HIGH (8)HIGH-VERY HIGH
(9)VERY HIGH/
PH (-0)UNKNOWN (01)BELOW 4.5 (02)4.5-5.0 (03)5.1-5.5 (04)5.6-6.
0 (05)6.1-6.5 (06)6.6-7.3 (07)7.4-7.8 (08)7.9-8.4 (09)8.5-9.0
(10)ABOVE 9.0/
LL,PL,PI,SL (-0)UNKNOWN (-1)NOT PLASTIC/
TEXTUR (-0)UNKNOWN (1)SAND (2)SAND-TRACE GRAVEL (3)SAND-LITTLE GR
AVEL (4)SAND-SOME GRAVEL (5)SAND AND GRAVEL (6)SANDY LOAM
(7)SANDY LOAM-TRACE GRAVEL (8)SANDY LOAM-LITTLE GRAVEL
(9)SANDY LOAM-SOME GRAVEL (10)SANDY LOAM AND GRAVEL
(11)LOAM (12)LOAM-TRACE GRAVEL (13)LOAM-LITTLE GRAVEL

(14)LOAM-SOME GRAVEL (15)LOAM AND GRAVEL (16)SILTY LOAM
(17)SILTY LOAM-TRACE GRAVEL (18)SILTY LOAM-LITTLE GRAVEL
(19)SILTY LOAM-SOME GRAVEL (20)SILTY LOAM AND GRAVEL
(21)SILT (22)SILT-TRACE GRAVEL (23)SILT-LITTLE GRAVEL
(24)SILT-SOME GRAVEL (25)SILT AND GRAVEL (26)SANDY CLAY LOAM
(27)SANDY CLAY LOAM-TRACE GRAVEL (28)SANDY CLAY LOAM-LITTLE GRAVE
L (29)SANDY CLAY LOAM-SOME GRAVEL (30)SANDY CLAY LOAM AND GRAVEL
(31)CLAY LOAM (32)CLAY LOAM-TRACE GRAVEL (33)CLAY LOAM-LITTLE GRA
VEL (34)CLAY LOAM-SOME GRAVEL (35)CLAY LOAM AND GRAVEL
(36)SILTY CLAY LOAM (37)SILTY CLAY LOAM-TRACE GRAVEL
(38)SILTY CLAY LOAM-LITTLE GRAVEL (39)SILTY CLAY LOAM-SOME GRAVEL
(40)SILTY CLAY LOAM AND GRAVEL (41)SANDY CLAY
(42)SANDY CLAY-TRACE GRAVEL (43)SANDY CLAY-LITTLE GRAVEL
(44)SANDY CLAY-SOME GRAVEL (45)SANDY CLAY AND GRAVEL
(46)SILTY CLAY (47)SILTY CLAY-TRACE GRAVEL (48)SILTY CLAY-LITTLE
GRAVEL (49)SILTY CLAY-SOME GRAVEL (50)SILTY CLAY AND GRAVEL
(51)CLAY (52)CLAY-TRACE GRAVEL (53)CLAY-LITTLE GRAVEL
(54)CLAY-SOME GRAVEL (55)CLAY AND GRAVEL
(56)GRAVEL (57)SANDY GRAVEL (58)GRAVELLY SAND
(59)SAND AND GRAVEL
(60)COPROGENOUS EARTH (61)DIATOMACEOUS EARTH (62)FIBRIC MATERIAL
(63)FRAGMENTAL MATERIAL (64)HEMIC MATERIAL (65)ICE OR FROZEN SOIL
(66)MARL (67)MUCK (68)MUCKY PEAT (69)OXIDE-PROTECT. WX BR
(70)PART. DECOM ORG MATL (71)PEAT (72)SAPRIC MATERIAL
(73)UNDECOM ORG MATL (74)UNWEATHERED BEDROCK
(75)WX BR,SAPROLITE,GRUS (76)COMPLEX (77)INAPPLICABLE/
ORGANI (-0)UNKNOWN (1)NO ORGANIC MATL (2)TRACE (3)LITTLE (4)SOME
(5)AND/
COLOR (-0)UNKNOWN (01)BLUISH GRAY (02)BLACK
(03)BROWN (04)BROWNISH YELLOW (05)DARK BROWN (06)DARK BLUISH GRAY
(07)DARK GRAY (08)DARK GRAYISH BROWN (09)DARK GREENISH GRAY
(10)DARK OLIVE (11)DARK OLIVE GRAY (12)DARK RED
(13)DARK REDDISH BROWN (14)DARK REDDISH GRAY (15)DUSKY RED
(16)DARK YELLOWISH BROWN (17)GRAYISH BROWN (18)GREENISH GRAY
(19)GRAYISH GREEN (20)GREEN (21)GRAY (22)LIGHT BLUISH GRAY
(23)LIGHT BROWN (24)LIGHT BROWNISH GRAY (25)LIGHT GREENISH GRAY
(26)LIGHT GRAY (27)LIGHT OLIVE BROWN (28)LIGHT OLIVE GRAY
(29)LIGHT RED (30)LIGHT REDDISH BROWN (31)LIGHT YELLOWISH BR.
(32)OLIVE BROWN (33)OLIVE (34)OLIVE GRAY (35)OLIVE YELLOW
(36)PALE BROWN (37)PALE GREEN (38)PALE OLIVE (39)PALE RED
(40)PALE YELLOW (41)PINKISH GRAY (42)PINK (43)PINKISH WHITE
(44)REDDISH BLACK (45)REDDISH BROWN (46)RED (47)REDDISH GRAY
(48)REDDISH YELLOW (49)STRONG BROWN (50)VERY DARK BROWN
(51)VERY DARK GRAY (52)V. DK. GRAYISH BROWN (53)VERY DARK RED
(54)VERY PALE BROWN (55)VERY DUSKY RED (56)WEAK RED (57)WHITE
(58)YELLOWISH BROWN (59)YELLOWISH RED (60)YELLOW/
TESTEF (-0)UNKNOWN (01)STANDARD-12400
(02)STANDARD-SEE DIF (03)STANDARD-NOT GIVEN (04)MODIFIED-12400
(05)MODIFIED-56000 (06)MODIFIED-56300 (07)MODIFIED-SEE DIF
(08)MODIFIED-NOT GIVEN (09)15 BLOW-7400 (10)15 BLOW-7800
(11)15 BLOW-SEE DIF (12)15 BLOW-NOT GIVEN (13)KNEADING-SEE DIF
(14)KNEADING-NOT GIVEN (15)HARVARD MIN-SEE DIF
(16)HARVARD MIN-NOT GIVEN (17)HVEEM-SEE DIF (18)HVEEM-NOT GIVEN
(19)VIBRATORY-SEE DIF (20)VIBRATORY-NOT GIVEN (21)COMPRESSION-SEE
DIF (22)COMPRESSION-NOT GIVEN/

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                TYPE (-0)UNKNOWN (1)UU TEST (2)CU TEST-UNSATURATED
                (3)CU TEST-SATURATED (4)DIRECT SHEAR (5)CD TEST/
                HOLENO,SAMPNO,DATEYR,DATEDA,TOWN,RANGE,SECTIO TO BORING,STATNO,
                OFFSET,LINE1,LINE2,LABNO TO SPT,BEDRKS TO WATERS,GRAD01 TO COLL,
                LOSSIG TO SPECGR,MAXDD TO QUSTR,QUSTA,STRENGTH TO CU
                (-0)UNKNOWN
MISSING VALUES  COUNTY TO QUSTR,QUSTA TO CU (-0)/
COMMENT         THE FOLLOWING SERIES OF ≠IF≠ STATEMENTS ARE TO GIVE THE CODED
                DESIGNATIONS OF ORGANIC CONTENTS
IF              (LOSSIG GT 0.001 AND LE 1) ORGANI=1
IF              (LOSSIG GT 1 AND LE 10) ORGANI=2
IF              (LOSSIG GT 10 AND LE 20) ORGANI=3
IF              (LOSSIG GT 20 AND LE 35) ORGANI=4
IF              (LOSSIG GT 35) ORGANI=5
COMMENT         THE FOLLOWING SERIES OF ≠IF≠ STATEMENTS ARE TO CORRECT THE
                PHYSIOGRAPHIC REGION PERTAINING TO A COUNTY
IF              (COUNTY EQ 65) PHYSIO=8
IF              (COUNTY EQ 27) PHYSIO=8
IF              (COUNTY EQ 79) PHYSIO=1
COMMENT         THE FOLLOWING SERIES OF ≠IF≠ STATEMENTS ARE TO GIVE CORRECT
                PARENT MATERIALS PERTAINING TO SOIL ASSOCIATIONS
IF              (ASSOC EQ 55) PARENT=2
IF              (ASSOC EQ 56) PARENT=2
IF              (ASSOC EQ 57) PARENT=2
IF              (ASSOC EQ 58) PARENT=5
IF              (ASSOC EQ 59) PARENT=7
IF              (ASSOC EQ 61) PARENT=6
IF              (ASSOC EQ 62) PARENT=6
IF              (ASSOC EQ 63) PARENT=5
IF              (ASSOC EQ 64) PARENT=7
IF              (ASSOC EQ 65) PARENT=6
IF              (ASSOC EQ 66) PARENT=8
IF              (ASSOC EQ 67) PARENT=6
IF              (ASSOC EQ 69) PARENT=7
IF              (ASSOC EQ 70) PARENT=7
IF              (ASSOC EQ 71) PARENT=7
IF              (ASSOC EQ 72) PARENT=8
IF              (ASSOC EQ 73) PARENT=8
IF              (ASSOC EQ 74) PARENT=8
IF              (ASSOC EQ 76) PARENT=8
IF              (ASSOC EQ 77) PARENT=8
IF              (ASSOC EQ 78) PARENT=7
IF              (ASSOC EQ 79) PARENT=5
IF              (ASSOC EQ 80) PARENT=5
IF              (ASSOC EQ 81) PARENT=8
IF              (ASSOC EQ 82) PARENT=7
IF              (ASSOC EQ 83) PARENT=7
IF              (ASSOC EQ 84) PARENT=7
IF              (ASSOC EQ 85) PARENT=2
IF              (ASSOC EQ 86) PARENT=6
IF              (ASSOC EQ 87) PARENT=7
IF              (ASSOC EQ 88) PARENT=7
IF              (ASSOC EQ 89) PARENT=7
IF              (ASSOC EQ 90) PARENT=7
IF              (ASSOC EQ 91) PARENT=9
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IF              (ASSOC EQ 92) PARENT=9
IF              (ASSOC EQ 93) PARENT=9
IF              (ASSOC EQ 95) PARENT=9
IF              (ASSOC EQ 96) PARENT=9
IF              (ASSOC EQ 98) PARENT=11
IF              (ASSOC EQ 99) PARENT=11
IF              (ASSOC EQ 100) PARENT=10
IF              (ASSOC EQ 101) PARENT=12
IF              (ASSOC EQ 102) PARENT=9
IF              (ASSOC EQ 103) PARENT=11
IF              (ASSOC EQ 104) PARENT=10
IF              (ASSOC EQ 105) PARENT=10
IF              (ASSOC EQ 106) PARENT=10
IF              (ASSOC EQ 107) PARENT=10
IF              (ASSOC EQ 108) PARENT=1
IF              (ASSOC EQ 109) PARENT=4
IF              (ASSOC EQ 110) PARENT=3
IF              (ASSOC EQ 111) PARENT=4
IF              (ASSOC EQ 112) PARENT=4
IF              (ASSOC EQ 113) PARENT=4
IF              (ASSOC EQ 114) PARENT=3
IF              (ASSOC EQ 115) PARENT=3
IF              (ASSOC EQ 116) PARENT=3
COMMENT         THE FOLLOWING SERIES OF ~IF~ STATEMENTS COMPUTE BOTH
                THE AASHTO AND UNIFIED CLASSIFICATION CODES-FOR THOSE
                PROGRAMS NOT REQUIRING EITHER OF THESE TWO CLASSIFICATIONS,
                THESE STATEMENTS SHOULD BE OMITTED
IF              (TEXTUR EQ 67 OR 68 OR 70 OR 71 OR 73 OR 60 OR 62) AASHTO=13
IF              (GRAD09 GE 35.5 AND LL GE 40.5 AND PI GE 10.5 AND PI GT (LL-30))
                AASHTO=12
IF              (GRAD09 GE 35.5 AND LL GE 40.5 AND PI GE 10.5 AND PI LE (LL-30))
                AASHTO=11
IF              (GRAD09 GE 35.5 AND LL LT 40.5 AND PI GE 10.5) AASHTO=10
IF              (GRAD09 GE 35.5 AND (LL GE 40.5 OR EQ -1) AND PI LT 10.5)
                AASHTO=09
IF              (GRAD09 GE 35.5 AND LL LT 40.5 AND PI LT 10.5) AASHTO=08
IF              (GRAD09 LT 35.5 AND LL GE 40.5 AND PI GE 10.5) AASHTO=06
IF              (GRAD09 LT 35.5 AND LL LT 40.5 AND PI GE 10.5) AASHTO=05
IF              (GRAD09 LT 35.5 AND (LL GE 40.5 OR EQ -1) AND PI LT 10.5)
                AASHTO=04
IF              (GRAD09 LT 35.5 AND LL LT 40.5 AND PI LT 10.5) AASHTO=03
IF              (GRAD08 GE 50.5 AND GRAD09 LT 10.5 AND (LL EQ -1 OR PL EQ -1
                OR PI EQ -1)) AASHTO=07
IF              (GRAD08 LT 50.5 AND GRAD09 LT 25.5 AND PI LT 6.5) AASHTO=02
IF              (GRAD07 LT 50.5 AND GRAD08 LT 30.5 AND GRAD09 LT 15.5
                AND PI LT 6.5) AASHTO=01
IF              (TEXTUR EQ 67 OR 68 OR 70 OR 71 OR 73 OR 60 OR 62) AASHTO=13
IF              (AASHTO NE 05 OR 06)
                GI=RND((GRAD09-35)*(.2+.005*(LL-40))+.01*(GRAD09-15)*(PI-10))
IF              (AASHTO EQ 05 OR 06) GI=RND(.01*(GRAD09-15)*(PI-10))
IF              (PL EQ -1 OR LL EQ -1 OR GI LT 0) GI=0
ASSIGN MISSING  GI (-8)
IF              (TEXTUR EQ 67 OR 68 OR 70 OR 71 OR 73 OR 60 OR 62) UNIF=35
IF              (GRAD09 GE 50 AND (PI GT (.73*(LL-20)) AND 7) AND LL LT 50)
                UNIF=01
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IF      (GRADO9 GE 50 AND PI GT (.73*(LL-20)) AND LL GT 50) UNIF=02
IF      (GRADO9 GE 50 AND PI GT (.73*(LL-20)) AND LL EQ 50) UNIF=03
IF      (GRADO9 GE 50 AND (PI LT (.73*(LL-20)) OR 4) AND LL LT 50 AND
        (ORGANI EQ 1 OR 2)) UNIF=04
IF      (GRADO9 GE 50 AND (PI LT (.73*(LL-20)) OR 4) AND LL LT 50 AND
        (ORGANI EQ 4 OR 5)) UNIF=05
IF      (GRADO9 GE 50 AND (PI LT (.73*(LL-20)) OR 4) AND LL LT 50 AND
        ORGANI EQ 3) UNIF=06
IF      (GRADO9 GE 50 AND (PI EQ (.73*(LL-20)) AND GT 7) AND LL LT 50 AND
        (ORGANI EQ 1 OR 2)) UNIF=07
IF      (GRADO9 GE 50 AND (PI EQ (.73*(LL-20)) AND GT 7) AND LL LT 50 AND
        (ORGANI EQ 4 OR 5)) UNIF=08
IF      (GRADO9 GE 50 AND (PI EQ (.73*(LL-20)) AND GT 7) AND LL LT 50 AND
        ORGANI EQ 3) UNIF=09
IF      (GRADO9 GE 50 AND PI LT (.73*(LL-20)) AND LL GT 50 AND
        (ORGANI EQ 1 OR 2)) UNIF=10
IF      (GRADO9 GE 50 AND PI LT (.73*(LL-20)) AND LL GT 50 AND
        (ORGANI EQ 4 OR 5)) UNIF=11
IF      (GRADO9 GE 50 AND PI LT (.73*(LL-20)) AND LL GT 50 AND ORGANI EQ
        3) UNIF=12
IF      (GRADO9 GE 50 AND PI EQ (.73*(LL-20)) AND LL GT 50 AND
        (ORGANI EQ 1 OR 2)) UNIF=13
IF      (GRADO9 GE 50 AND PI EQ (.73*(LL-20)) AND LL GT 50 AND
        (ORGANI EQ 4 OR 5)) UNIF=14
IF      (GRADO9 GE 50 AND PI EQ (.73*(LL-20)) AND LL GT 50 AND ORGANI EQ
        3) UNIF=15
IF      (GRADO9 GE 50 AND PI LT (.73*(LL-20)) AND LL EQ 50 AND
        (ORGANI EQ 1 OR 2)) UNIF=16
IF      (GRADO9 GE 50 AND PI LT (.73*(LL-20)) AND LL EQ 50 AND
        (ORGANI EQ 4 OR 5)) UNIF=17
IF      (GRADO9 GE 50 AND PI LT (.73*(LL-20)) AND LL EQ 50 AND ORGANI EQ
        3) UNIF=18
IF      (GRADO9 GE 50 AND PI EQ (.73*(LL-20)) AND LL EQ 50) UNIF=19
IF      (GRADO9 GE 50 AND (PI GE (.73*(LL-20)) AND LE 7 AND GE 4))
        UNIF=20
IF      (GRADO9 LT 50 AND GRADO6 LE 50 AND GRADO9 GT 12 AND (PI LT
        (.73*(LL-20)) OR 4)) UNIF=21
IF      (GRADO9 LT 50 AND GRADO6 GT 50 AND GRADO9 GT 12 AND (PI LT
        (.73*(LL-20)) OR 4)) UNIF=24
IF      (GRADO9 LT 50 AND GRADO6 LE 50 AND GRADO9 GT 12 AND (PI GT
        (.73*(LL-20)) AND 7)) UNIF=22
IF      (GRADO9 LT 50 AND GRADO6 GT 50 AND GRADO9 GT 12 AND (PI GT
        (.73*(LL-20)) AND 7)) UNIF=25
IF      (GRADO9 LT 50 AND GRADO6 LE 50 AND GRADO9 GT 12 AND ((PI EQ
        (.73*(LL-20))) OR (PI GT (.73*(LL-20)) AND LE 7 AND GE 4)))
        UNIF=23
IF      (GRADO9 LT 50 AND GRADO6 GT 50 AND GRADO9 GT 12 AND ((PI EQ
        (.73*(LL-20))) OR (PI GT (.73*(LL-20)) AND LE 7 AND GE 4)))
        UNIF=26
IF      (GRADO9 LT 50 AND GRADO6 LE 50 AND GRADO9 LT 5) UNIF=27
IF      (GRADO9 LT 50 AND GRADO6 GT 50 AND GRADO9 LT 5) UNIF=28
IF      (GRADO9 LT 50 AND GRADO6 LE 50 AND (GRADO9 GE 5 AND LE 12) AND
        (PI LT (.73*(LL-20)) OR 4)) UNIF=29
IF      (GRADO9 LT 50 AND GRADO6 LE 50 AND (GRADO9 GE 5 AND LE 12) AND
        (PI GT (.73*(LL-20)) AND 7)) UNIF=30
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IF                (GRAD09 LT 50 AND GRAD06 LE 50 AND (GRAD09 GE 5 AND LE 12) AND
                  ((PI EQ (.73*(LL-20))) OR (PI GT (.73*(LL-20)) AND LE 7 AND
                  GE 4))) UNIF=31
IF                (GRAD09 LT 50 AND GRAD06 GT 50 AND (GRAD09 GE 5 AND LE 12) AND
                  (PI LT (.73*(LL-20)) OR 4)) UNIF=32
IF                (GRAD09 LT 50 AND GRAD06 GT 50 AND (GRAD09 GE 5 AND LE 12) AND
                  (PI GT (.73*(LL-20)) AND 7)) UNIF=33
IF                (GRAD09 LT 50 AND GRAD06 GT 50 AND (GRAD09 GE 5 AND LE 12) AND
                  ((PI EQ (.73*(LL-20))) OR (PI GT (.73*(LL-20)) AND LE 7 AND
                  GE 4))) UNIF=34
IF                (TEXTUR EQ 67 OR 68 OR 70 OR 71 OR 73 OR 60 OR 62) UNIF=35
IF                (AASHTO1 NE 9999) AASHTO=AASHTO1
IF                (UNIF1 NE 9999) UNIF=UNIF1
IF                (SAND EQ 999) SAND=-0
IF                (SAND NE 999) SAND=GRAD07-GRAD09
ASSIGN MISSING    AASHTO,UNIF (-8)
VAR LABELS        GI GROUP INDEX/AASHTO AASHTO CLASSIFICATION/UNIF UNIFIED CLASSIFI
                  CATION
VALUE LABELS      GI (-8)MISSING DATA/
                  AASHTO (-0)UNKNOWN (1)A-1-A (2)A-1-B (3)A-2-4 (4)A-2-5 (5)A-2-6
                  (6)A-2-7 (7)A-3 (8)A-4 (9)A-5 (10)A-6 (11)A-7-5 (12)A-7-6
                  (13)A-8/
                  UNIF (-0)UNKNOWN (01)CL (02)CH (03)CL-CH (04)ML (05)OL (06)ML-OL
                  (07)ML-CL (08)CL-OL (09)ML-OL OR CL (10)MH (11)OH (12)MH-OH
                  (13)MH-CH (14)CH-OH (15)MH-OH OR  CH (16)ML-MH (17)OL-OH
                  (18)MH-OH OR ML-OL (19)ANY COMBINATION (20)CL-ML
                  (21)GM (22)GC (23)GM-GC (24)SM (25)SC (26)SM-SC (27)GW OR GP*=G
                  (28)SW OR SP*=S (29)G-GM (30)G-GC (31)G-GM OR G-GC (32)S-SM
                  (33)S-SC (34)S-SM OR S-SC (35)PT
COMMENT           THE ABOVE STATEMENT IS THE LAST STATEMENT WHICH
                  SHOULD BE OMITTED IF THE AASHTO AND UNIFIED
                  CLASSIFICATIONS ARE NOT REQUIRED
COMMENT           THE NUMBER OF CASES (N OF CASES) WILL COINCIDE WITH
                  THE NUMBER OF SAMPLES CONTAINED WITHIN THE DATA BANK
                  AT THE TIME OF PROCESSING
N OF CASES        9442
PRINT FORMATS     SPECGR,EO,EF,CC,CR (3)/  QUSTR,QUSTA,STRENGTH TO COHESION,
                  POREPRESS,MAJOR,PO,PC,CV (2)/ GRDSUR TO DEPTHB,
                  BEDRKS TO WATERF,
                  GRAD01 TO NATDD,MAXDD TO CBRSO2,ANGLE,SO,SF (1)/PROJPR,
                  PROJPA,CONTPR,ROADPR,ROADSU TO BORING;LINE1,LINE2,LABNO (A)
READ INPUT DATA
SAVE FILE         PROG
FINISH
6/7/8/9
```

APPENDIX B-II

DATA RETRIEVAL PROGRAMS

1. The following program is to retreive the geotechnical information in the area of Township 37 North (T37N) and Range 9 West (R9W) then to put it onto the file in terms of PFILES VIKING 8

```
12345,ABC,CM150000,L10000,T400,TU30000,TC300,F20,TP1.
PASS=CDEF
REQUEST(TAPE,123)
REWIND(TAPE)
COPYBF(TAPE,PROG)
RETURN(TAPE)
REWIND(PROG)
COMMON(SPSS)
SPSS(G=PROG,A=VIKING3)
REWIND(VIKING3)
PFILES(PUT,VIKING3)
7/8/9
GET FILE        PROG
SELECT IF       (TOWN EQ 37 AND TOWNDI EQ 1 AND RANGE EQ 9 AND RANGDI EQ 2)
WRITE CASES     (1X,F2.0,F5.0,F2.0,1X,F1.0,1X,F2.0,1X,F2.0, 1X,F2.0, 1X,F2.0,
                1X,F1.0,1X,F2.0, 1X,F1.0,1X,F2.0,1X,A3,  A5,A3,F3.0,1X,A3,
                F5.0, 1X,A2,F3.0,A1, 1X,A3, 1X,F3.0, 1X,F1.0/
                11X,F7.0, 1X,F4.0, 1X,F1.0, 1X, A3,A2, 1X,F2.0, 1X,F2.0, 1X, A3,
                1X,F5.1,   1X,F4.1, 1X,F4.1, 1X,F2.0, 1X,F2.0, 1X,F4.0/
                11X,F2.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F3.1, 1X,F4.1, 1X,F2.1,
                1X,F4.1, 1X,F4.1, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0,
                1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1/ 11X,
                F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1.
                1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1/
                11X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.3, 1X,F2.0, 1X,F1.0, 1X,F2.0,
                1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1, 1X,F3.1, 1X,F3.1,
                1X,F3.1, 1X,F4.2,F2.0,F2.0/ 11X F4.2,1X,F1.0,3F4.2,F3.2,
                F3.1,2F4.2,1X,2F4.3,2F4.1,2F4.2,2F4.3,F4.2)
                COUNTY,HOLENO,SAMPNO,DISTRI,DATEYR,DATEMO,DATEDA,TOWN,TOWNDI,
                RANGE,RANGDI,SECTIO,PROJPR,PROJNO,PROJPA,PROJMI,CONTPR,
                CONTNO,ROADPR,ROADNO,ROADSU,BORING,ASSOC,REPEAT,STATNO,OFFSET,
                OFFDIR,LINE1,LINE2,SOURCE,SAMPTY,LABNO,GRDSUR,DEPTHT,
                DEPTHB,SPT,PHYSIO,SERIES,PARENT,HORIZO,SLOPE,EROSIO,BEDRKS,
                BEDRKB,WATERS,WATERC,WATERF,DRAIN,PERMEA,FLOOD,FROST,SHRINK,PH,
                GRAD01 TO GRAD10,SAND,SILT,CLAY,COLL,LL,PL,PI,SL,LOSSIG,
                NATMC,NATWD,NATDD,SPECGR,TEXTUR,ORGANI,COLOR,TESTEF,MAXDD,
                MAXWD,OPTIMC,CBRUN1,CBRUN2,CBRSO1,CBRSO2,QUSTR,AASHTO,UNIF,
                QUSTA,TYPE,STRENGTH,STRAIN,CONFPRES,COHESION,ANGLE,POREPRES,
                MAJOR,EO,EF,SO,SF,PO,PC,CC,CR,CV
FINISH
```

2. The following program is to present one-way frequency distribution tables for the variables as indicated in the program with the information out of PFILES VIKING 8

```
12345,ABC,MF150000,CM150000,L10000,P20,TU48000,TC480,T400.
PASS=CDEF
PFILES(GET,VIKING8)
COMMON(SPSS)
SPSS(D=VIKING8)
7/8/9
VARIABLE LIST   COUNTY,HOLENO,SAMPNO,DISTRI,DATEYR,DATEMO,DATEDA,TOWN,TOWNDI,
                RANGE,RANGDI,SECTIO,PROJPR,PROJNO,PROJPA,PROJMI,CONTPR,
                CONTNO,ROADPR,ROADNO,ROADSU,BORING,ASSOC,REPEAT,STATNO,OFFSET,
                OFFDIR,LINE1,LINE2,SOURCE,SAMPTY,LABNO,GRDSUR,DEPTHT,
                DEPTHB,SPT,PHYSIO,SERIES,PARENT,HORIZO,SLOPE,EROSIO,BEDRKS,
                BEDRKB,WATERS,WATERC,WATERF,DRAIN,PERMEA,FLOOD,FROST,SHRINK,PH,
                GRAD01 TO GRAD10,SAND,SILT,CLAY,COLL,LL,PL,PI,SL,LOSSIG,
                NATMC,NATWD,NATDD,SPECGR,TEXTUR,ORGANI,COLOR,TESTEF,MAXDD,
                MAXWD,OPTIMC,CBRUN1,CBRUN2,CBRSO1,CBRSO2,QUSTR,AASHTO,UNIF,
                QUSTA,TYPE,STRENGTH,STRAIN,CONFPRES,COHESION,ANGLE,POREPRES,
                MAJOR,EO,EF,SO,SF,PO,PC,CC,CR,CU
N OF CASES      138
INPUT FORMAT    (1X,F2.0,F5.0,F2.0,1X,F1.0,1X,F2.0,1X,F2.0, 1X,F2.0, 1X,F2.0,
                1X,F1.0,1X,F2.0, 1X,F1.0,1X,F2.0,1X,A3,  A5,A3,F3.0,1X,A3,
                F5.0, 1X,A2,F3.0,A1, 1X,A8, 1X,F3.0, 1X,F1.0/
                11X,F7.0, 1X,F4.0, 1X,F1.0, 1X, A3,A2, 1X,F2.0, 1X,F2.0, 1X, A8,
                1X,F5.1,   1X,F4.1, 1X,F4.1, 1X,F2.0, 1X,F2.0, 1X,F4.0/
                11X,F2.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F3.1, 1X,F4.1, 1X,F2.1,
                1X,F4.1, 1X,F4.1, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0,
                1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1/ 11X,
                F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1,
                1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1/
                11X,F4.1, 1X,F4.1, 1X,F4.1, 1X.F4.3, 1X,F2.0, 1X,F1.0, 1X,F2.0,
                1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1, 1X,F3.1, 1X,F3.1,
                1X,F3.1, 1X,F4.2,F2.0,F2.0/ 11X F4.2,1X,F1.0,3F4.2,F3.2,
                F3.1,2F4.2,1X,2F4.3,2F4.1,2F4.2,2F4.3,F4.2)
FREQUENCIES     GENERAL=SECTIO,ASSOC,GRDSUR,DEPTHT,DEPTHB,SPT,PHYSIO,SERIES,
                PARENT,BEDRKS,BEDRKB,WATERF,PH,SAND,SILT,CLAY,COLL,LL,PL,PI,
                SL,NATMC,NATDD,SPECGR,TEXTUR,ORGANI,MAXDD,OPTIMC,CBRSO1,CBRSO2,
                QUSTR,AASHTO,UNIF,COHESION,ANGLE,EO,PO,PC,CC,CR,CU
OPTIONS         5
FINISH
```

3. The following program is to present the distributions of textures, AASHTO classification units, Unified classification units, SPT values, and unconfined compressive strength values vs. depth with the information out of PFILES VIKING 8

```
12345,ABC,MF150000,CM150000,L10000,P20,TU48000,TC480,T400.
PASS=CDEF
PFILES(GET,VIKING8)
COMMON(SPSS)
SPSS(D=VIKING8)
COPYCF(BCDOUT,,,RIB)
7/8/9
VARIABLE LIST   COUNTY,HOLENO,SAMPNO,DISTRI,DATEYR,DATEMO,DATEDA,TOWN,TOWNDI,
                RANGE,RANGDI,SECTIO,PROJPR,PROJNO,PROJPA,PROJMI,CONTPR,
                CONTNO,ROADPR,ROADNO,ROADSU,BORING,ASSOC,REPEAT,STATNO,OFFSET,
                OFFDIR,LINE1,LINE2,SOURCE,SAMPTY,LABNO,GRDSUR,DEPTHT,
                DEPTHB,SPT,PHYSIO,SERIES,PARENT,HORIZO,SLOPE,EROSIO,BEDRKS,
                BEDRKB,WATERS,WATERC,WATERF,DRAIN,PERMEA,FLOOD,FROST,SHRINK,PH,
                GRAD01 TO GRAD10,SAND,SILT,CLAY,COLL,LL,PL,PI,SL,LOSSIG,
                NATMC,NATWD,NATDD,SPECGR,TEXTUR,ORGANI,COLOR,TESTEF,MAXDD,
                MAXWD,OPTIMC,CBRUN1,CBRUN2,CBRS01,CBRS02,QUSTR,AASHTO,UNIF,
                QUSTA,TYPE,STRENGTH,STRAIN,CONFPRES,COHESION,ANGLE,POREPRES,
                MAJOR,EO,EF,SO,SF,PO,PC,CC,CR,CV
N OF CASES      138
INPUT FORMAT    (1X,F2.0,F5.0,F2.0,1X,F1.0,1X,F2.0,1X,F2.0, 1X,F2.0, 1X,F2.0,
                1X,F1.0,1X,F2.0, 1X,F1.0,1X,F2.0,1X,A3,  A5,A3,F3.0,1X,A3,
                F5.0, 1X,A2,F3.0,A1, 1X,A8, 1X,F3.0, 1X,F1.0/
                11X,F7.0, 1X,F4.0, 1X,F1.0, 1X, A8,A2, 1X,F2.0, 1X,F2.0, 1X, A8,
                1X,F5.1,   1X,F4.1, 1X,F4.1, 1X,F2.0, 1X,F2.0, 1X,F4.0/
                11X,F2.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F3.1, 1X,F4.1, 1X,F2.1,
                1X,F4.1, 1X,F4.1, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0,
                1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1/ 11X,
                F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1,
                1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1/
                11X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.3, 1X,F2.0, 1X,F1.0, 1X,F2.0,
                1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1, 1X,F3.1, 1X,F3.1,
                1X,F3.1, 1X,F4.2,F2.0,F2.0/ 11X F4.2,1X,F1.0,3F4.2,F3.2,
                F3.1,2F4.2,1X,2F4.3,2F4.1,2F4.2,2F4.3,F4.2)
COMPUTE         DEPTH=(DEPTHT + DEPTHB)/2
SORT CASES      DEPTH(A)
WRITE CASES     (F6.2,5X,F2.0,5X,F2.0,5X,F2.0,5X,F2.0,5X,F5.2)DEPTH,TEXTUR,
                AASHTO,UNIF,SPT,QUSTR
FINISH
```

4. The following program is to present the CALCOMP plots of liquid limits, plastic limits, and natural moisture contents vs. depths and SPT values vs. depths with the information out of PFILES VIKING 8

```
12345,ABC,MF150000,CM150000,L10000,P20,TU48000,TC480,T400.
PASS=CDEF
PFILES(GET,VIKING8)
COMMON(SPSS)
SPSS(D=VIKING8)
COPYPLT(NOPLOT)
7/8/9
RUN NAME        WILD GEESE
VARIABLE LIST   COUNTY,HOLENO,SAMPNO,DISTRI,DATEYR,DATEMO,DATEDA,TOWN,TOWNDI,
                RANGE,RANGDI,SECTIO,PROJPR,PROJNO,PROJPA,PROJMI,CONTPR,
                CONTNO,ROADPR,ROADNO,ROADSU,BORING,ASSOC,REPEAT,STATNO,OFFSET,
                OFFDIR,LINE1,LINE2,SOURCE,SAMPTY,LABNO,GRDSUR,DEPTHT,
                DEPTHB,SPT,PHYSIO,SERIES,PARENT,HORIZO,SLOPE,EROSIO,BEDRKS,
                BEDRKB,WATERS,WATERC,WATERF,DRAIN,PERMEA,FLOOD,FROST,SHRINK,PH,
                GRAD01 TO GRAD10,SAND,SILT,CLAY,COLL,LL,PL,PI,SL,LOSSIG,
                NATMC,NATWD,NATDD,SPECGR,TEXTUR,ORGANI,COLOR,TESTEF,MAXDD,
                MAXWD,OPTIMC,CBRUN1,CBRUN2,CBRSO1,CBRSO2,QUSTR,AASHTO,UNIF,
                QUSTA,TYPE,STRENGTH,STRAIN,CONFPRES,COHESION,ANGLE,POREPRES,
                MAJOR,EO,EF,SO,SF,PO,PC,CC,CR,CV
N OF CASES      138
INPUT FORMAT    (1X,F2.0,F5.0,F2.0,1X,F1.0,1X,F2.0,1X,F2.0, 1X,F2.0, 1X,F2.0,
                1X,F1.0,1X,F2.0, 1X,F1.0,1X,F2.0,1X,A3,  A5,A3,F3.0,1X,A3,
                F5.0, 1X,A2,F3.0,A1, 1X,A8, 1X,F3.0, 1X,F1.0/
                11X,F7.0, 1X,F4.0, 1X,F1.0, 1X, A8,A2, 1X,F2.0, 1X,F2.0, 1X, A8,
                1X,F5.1,   1X,F4.1, 1X,F4.1, 1X,F2.0, 1X,F2.0, 1X,F4.0/
                11X,F2.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F3.1, 1X,F4.1, 1X,F2.1,
                1X,F4.1, 1X,F4.1, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0, 1X,F1.0,
                1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1/ 11X,
                F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1,
                1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1/
                11X,F4.1, 1X,F4.1, 1X,F4.1, 1X,F4.3, 1X,F2.0, 1X,F1.0, 1X,F2.0,
                1X,F2.0, 1X,F4.1, 1X,F4.1, 1X,F3.1, 1X,F3.1, 1X,F3.1, 1X,F3.1,
                1X,F3.1, 1X,F4.2,F2.0,F2.0/ 11X F4.2,1X,F1.0,3F4.2,F3.2,
                F3.1,2F4.2,1X,2F4.3,2F4.1,2F4.2,2F4.3,F4.2)
COMPUTE         DEPTH=(DEPTHT + DEPTHB)/2
PLOT            PLOTS=LL,PL,NATMC(0.001,60) WITH DEPTH(0,140)/
                SIZE=-7.5,-4/
                XDIV=10/YDIV=6/
                SYMBOLS=1,2,0/
                PLOTS=SPT(0.001,80) WITH DEPTH(0,140)/
                SIZE=-7.5,-4/
                XDIV=10/YDIV=5/
                SYMBOLS=3/
FINISH
```

APPENDIX C: MULTIPLE COMPARISON TABLES

Note:

*VARIABLE NAME PHYSIO

CODE	DESCRIPTION
1.	Tipton Till Plain
2.	Dearborn Upland
3.	Muscatatuck Regional Slope
4.	Scottsburg Lowland
5.	Norman Upland
6.	Mitchell Plain
7.	Crawford Upland
8.	Wabash Lowland
9.	Calumet Lacustrine Section
10.	Valparaiso Moraine
11.	Kankakee Lacustrine Section
12.	Maumee Lacustrine Section
13.	Steuben Morainal Section

**VARIABLE NAME AASHTO

CODE	DESCRIPTION
1.	A-1-A
2.	A-1-B
3.	A-2-4
4.	A-2-5
5.	A-2-6
6.	A-2-7
7.	A-3
8.	A-4
9.	A-5
10.	A-6
11.	A-7-5
12.	A-7-6
13.	A-8

***VARIABLE NAME ASSOC

CODE	DESCRIPTION
1.	Eel-Martinsville-Genesee
2.	Genesee-Ross-Shoals
3.	Wakeland-Stendal-Haymond-Bartle
4.	Genesee-Shoals-Eel
5.	Haymond-Nolin-Petrolia
6.	Genesee-Eel-Stendal-Pope
7.	Huntington-Wheeling-Markland
8.	Huntington-Lindside
9.	Haymond-Wakeland
10.	Alida-Del Rey-Whitaker
11.	Bono-Maumee-Warners
12.	Chelsea-Hillsdale-Oshtemo
13.	Conrad-Wooten-Weiss
14.	Door-Tracy-Quinn
15.	Door-Lydick
16.	Elston-Wea
17.	Dubois-Otwell-Bartle
18.	Fox-Martinsville-Aluvi
19.	Fox-Nineveh-Ockley
20.	Fox-Rodman
21.	Fulton-Rimer-Milford-Rensselaer
22.	Homer-Sebewa-Gilford
23.	Maumee-Gilford-Rensselaer
24.	Maumee-Newton
26.	Martinsville-Bellmore-Fox
27.	Martinsville-Whitaker
28.	Mahalasville-Whitaker
30.	Milford-Montgomery-Rensselaer
31.	McGary
32.	Negley-Parke
33.	Oshtemo-Bronson
34.	Oakville-Plainfield-Adrian
35.	Oshtemo-Fox

***VARIABLE NAME ASSOC (continued)

CODE	DESCRIPTION
36.	Ockley-Westland
37.	Ockley-Wea
38.	Ockley-Fox
39.	Plainfield-Brems-Morrocco
40.	Plainfield-Tyner-Oshtemo
41.	Plainfield-Watseka
42.	Plainfield-Chelsea
43.	Patton-Henshaw
44.	Patton-Lyles-Henshaw
45.	Peoga-Bartle-Hosmer
46.	Parke-Miami-Negley
47.	Rensselaer-Montgomery
48.	Rensselaer-Darroch
49.	Rensselaer-Whitaker
50.	Vincennes-Zipp-Ross
51.	Volinia
53.	Wea-Crane
54.	Warsaw-Elston-Fox
55.	Westland-Sleeth
56.	Weinbach-Sciotoville
57.	Weinbach-Wheeling
58.	Crosier-Brookston
59.	Brookston-Odell-Corwin
61.	Blount-Morley-Pewamo
62.	Blount-Pewamo
63.	Riddles-Miami-Crosier
64.	Crosby-Brookston
65.	Elliott-Markham-Pewamo
66.	Fincastle-Ragsdale-Brookston
67.	Hoytville-Nappanee
69.	Parr-Miami
70.	Parr-Corwin

***VARIABLE NAME ASSOC (continued)

CODE	DESCRIPTION
71.	Randolph-Millsdale
72.	Reesville-Ragsdale
73.	Raub-Ragsdale
74.	Ragsdale-Sidell
76.	Russell-Hennepin
77.	Russell-Xenia
78.	Miami-Metea-Celina
79.	Miami-Owosso-Riddles
80.	Miami-Crosier-Metea
81.	Miami-Russell-Fincastle
82.	Miami-Fox-Milton
83.	Miami-Crosby
84.	Miami-Hennepin
85.	Miami-Fox-Martinsville
86.	Morley-Blount
87.	Muskingum-Shadeland-High Gap
88.	Odell-Chalmers
89.	Sidell-Parr
90.	Hennepin-Rodman
91.	Avonburg-Clermont
92.	Cincinnati-Hickory
93.	Cincinnati-Rossmoyne-Hickory
95.	Cincinnati-Ava
96.	Cincinnati-Ava-Alford
98.	Crider-Hagerstown-Frederick
99.	Crider-Frederick
100.	Corydon-Weikert-Berks
101.	Fairmount-Switzerland
102.	Grayford
103.	Lawrence-Bedford-Crider
104.	Tilsit-Johnsburg
105.	Wellston-Zanesville-Berks

***VARIABLE NAME ASSOC (continued)

CODE	DESCRIPTION
106.	Berks-Gilpin-Weikert
107.	Zanesville-Wellston
108.	Mucks-Peats
109.	Alford
110.	Bloomfield-Princeton-Ayrshire
111.	Hosmer
112.	Iva-Ava
113.	Hosmer-Cincinnati-Iva
114.	Lyles-Ayrshire-Princeton
115.	Princeton-Ayrshire-Bloomfield
116.	Princeton-Fox

****VARIABLE NAME SERIES

CODE	DESCRIPTION
10.	Ade
20.	Adrian
30.	Alford
40.	Algiers
50.	Alida
60.	Allison
70.	Armiesburg
80.	Aubbeenaubbee
90.	Ava
100.	Avonburg
110.	Ayr
120.	Ayrshire
130.	Bartle
140.	Baxter
150.	Bedford
160.	Bellmore
170.	Berks

****VARIABLE NAME SERIES (continued)

CODE	DESCRIPTION
180.	Birds
190.	Bloomfield
200.	Blount
210.	Bonnie
220.	Bono
230.	Boonesboro
240.	Boyer
250.	Brady
260.	Brems
270.	Bronson
280.	Brookston
290.	Burgin
300.	Burnside
310.	Camden
320.	Carlisle
330.	Casco
340.	Catlin
350.	Celina
360.	Chalmers
370.	Chelsea
380.	Cincinnati
390.	Clarence
400.	Clermont
410.	Colyer
420.	Conover
430.	Conrad
440.	Corwin
450.	Cory
460.	Corydon
470.	Coupee
480.	Crane
490.	Crider

****VARIABLE NAME SERIES (continued)

CODE	DESCRIPTION
500.	Crosby
510.	Crosier
520.	Cuba
530.	Dana
540.	Darroch
550.	Del Rey
560.	Dickinson
570.	Door
580.	Dowagiac
590.	Dubois
600.	Dunning
610.	Eden
620.	Edenton
630.	Edwards
640.	Eel
650.	Elkinsonville
660.	Elliott
670.	Elston
680.	Evansville
690.	Fabius
700.	Fairmount
710.	Fincastle
720.	Flanagan
730.	Foresman
740.	Fox-Silt Loam
741.	Fox-Loam
742.	Fox-Urban Land
750.	Frederick
760.	Fulton
770.	Genesee-Silt Loam
771.	Genessee-Urban Land
780.	Gilford
790.	Gilpin

****VARIABLE NAME SERIES (continued)

CODE	DESCRIPTION
800.	Ginat
810.	Glenhall
820.	Granby
830.	Grayford
840.	Guthrie
850.	Hagerstown
860.	Hanna
870.	Haskins
880.	Haubstadt
890.	Haymond
900.	Hennepin
910.	Henshaw
920.	Hickory
930.	High Gap
940.	Hillsdale
950.	Homer
960.	Hoopeston
970.	Hosmer
980.	Houghton
990.	Hoytville
1000.	Huntington
1010.	Huntsville
1020.	Iona
1030.	Ipava
1040.	Iva
1050.	Jasper
1060.	Jennings
1070.	Johnsburg
1080.	Jules
1090.	Kalamazoo
1100.	Kerston
1110.	Kings

****VARIABLE NAME SERIES (continued)

CODE	DESCRIPTION
1120.	Kokomo
1130.	Landes
1140.	Lawrence
1150.	Lenawee
1160.	Lindside
1170.	Linkville
1180.	Longlois
1190.	Lorenzo
1200.	Lowell
1210.	Lucas
1220.	Lydick
1230.	Lyles
1240.	Mahalasville
1250.	Markham
1260.	Markland
1270.	Martinsville
1280.	Martisco
1290.	Massie
1300.	Matherton
1310.	Maumee
1320.	McGary
1330.	Medway
1340.	Mellott
1350.	Mermill
1360.	Metamora
1370.	Metea
1380.	Miami-Silt Loam
1381.	Miami-Urban Land
1390.	Milford
1400.	Millsdale
1410.	Milton
1420.	Monitor

****VARIABLE NAME SERIES (continued)

CODE	DESCRIPTION
1430.	Montgomery
1440.	Montmorenci
1450.	Morley
1460.	Morocco
1470.	Muren
1480.	Muskingum
1490.	Mussey
1500.	Nappanee
1510.	Negley
1520.	Newark
1530.	Newton
1540.	Nicholson
1550.	Nineveh
1560.	Nolin
1570.	Oakville
1580.	Ockley
1590.	Octagon
1600.	Odell
1610.	Oshtemo
1620.	Otwell
1630.	Owosso
1640.	Palms
1650.	Parke
1660.	Parr
1670.	Patton
1680.	Pekin
1690.	Peoga
1700.	Petrolia
1710.	Pewamo
1720.	Pike
1730.	Pinhook
1740.	Plainfield

****VARIABLE NAME SERIES (continued)

CODE	DESCRIPTION
1750.	Plano
1760.	Pope
1770.	Princeton
1780.	Proctor
1790.	Quinn
1800.	Ragsdale
1810.	Rahm
1820.	Randolph
1830.	Rarden
1840.	Raub
1850.	Rawson
1860.	Reesville
1870.	Rensselaer
1880.	Riddles
1890.	Rimer
1900.	Robinson
1910.	Rockcastle
1920.	Rodman
1930.	Ross
1940.	Rossmoyne
1950.	Runnymede
1960.	Rush
1970.	Russell
1980.	St. Clair
1990.	Saranac
2000.	Saugatuck
2010.	Sciotoville
2020.	Sebewa
2030.	Seward
2040.	Shadeland
2050.	Shipshe
2060.	Shoals

****VARIABLE NAME SERIES (continued)

CODE	DESCRIPTION
2070.	Sidell
2080.	Sleeth
2090.	Sloan
2100.	Sparta
2120.	Starks
2120.	Steff
2130.	Stendal
2140.	Stonelick
2150.	Stoy
2160.	Strole
2170.	Sunbury
2180.	Switzerland
2190.	Swygert
2200.	Sylvan
2210.	Taggart
2220.	Tama
2230.	Tedrow
2240.	Tilsit
2250.	Tippecanoe
2260.	Toledo
2270.	Toronto
2280.	Tracy
2290.	Trappist
2300.	Troxel
2310.	Tyner
2320.	Uniontown
2330.	Vigo
2340.	Vincennes
2350.	Volinia
2360.	Wakeland
2370.	Wallkill
2380.	Warners
2390.	Warsaw

****VARIABLE NAME SERIES (continued)

CODE	DESCRIPTION
2400.	Wasepi
2410.	Washtenaw
2420.	Watseka
2430.	Wauseon
2440.	Wea
2450.	Weikert
2460.	Weinbach
2470.	Wellston
2480.	Westland
2490.	Wheeling
2500.	Whitaker
2510.	Whitson
2520.	Wilbur
2530.	Willette
2540.	Wingate
2550.	Woodmere
2560.	Woolper
2570.	Wynn
2580.	Xenia
2590.	Zanesville
2600.	Zipp
2700.	Borrow Pit
2800.	Urban Land
2900.	Alluvial Land
3000.	Gravel Pit
3100.	Gullied Land
3200.	Strip Mine
3300.	Cut and Fill
3400.	Made Land

CONTRA
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	CAL
No.	WORK ITEM
1	Move In
2	Remove Existing Struc
3	Order and Deliver Piling
4	Construct Fill
5	Bent 1 Cofferdam
6	Bent 1 Piling
7	Bent 1 Form & Pour Footi
	Bent 1

JHRP 82/7 3 Sheets

JHRP 80/7 Sheet 1

INDIANA STATE HIGHWAY COMMISSION

CONTRACT Bridge Project - Sheet 1
PROJECT -
DISTRICT -

LETTING DATE
START DATE May 1, 1980
COMPLETION DATE

MADE BY
CHECKED BY
APPROVAL BY

CALENDAR DAYS → 5/1/80 ... 9/30/80

WORK DAYS → 10 20 30 40 50 60 70 80 90 100

PROPOSED
ACTUAL

No.	WORK ITEMS	QUANTITY	DAILY PRODUCTIVITY	
1	Move In	-	-	
2	Remove Existing Structure	-	-	1
3	Order and Deliver Piling	-	-	
4	Construct Fill	8,000 cyd	500 cyd/day	2
5	Bent 1 Cofferdam	1 ea.		4
6	Bent 1 Piling	1500 lft	500 lft/day	3, 5
7	Bent 1 Form & Pour Footing	10 cyd	10 cyd/day	6
8	Bent 1 Cure Footing	-	-	7
9	Dewater, Form & Pour Bent 1 Stem	20 cyd	10 cyd/day	8
10	Bent 1 Cap	10 cyd	10 cyd/day	9
11	Bent 2 Cofferdam	1 ea.		5
12	Bent 2 Piling	1500 lft	500 lft/day	6, 11
13	Bent 2 Form & Pour Footing	10 cyd	10 cyd/day	7,9,12
14	Bent 2 Cure Footing	-	-	13
15	Dewater, Form & Pour Bent 2 Stem	20 cyd	10 cyd/day	9, 14
16	Bent 2 Cap	10 cyd	10 cyd/day	15
17	North End Bent Drive Piling	1000 lft	500 lft/day	12
18	North End Bent Form & Pour	30 cyd	10 cyd/day	15, 17
19	North End Bent Cure	-	-	18
20	South End Bent Drive Piling	100 lft	500 lft/day	17

JHRP 80/7 Sheet 2

INDIANA STATE HIGHWAY COMMISSION

CONTRACT Bridge Project - Sheet 2
PROJECT
DISTRICT

LETTING DATE
START DATE
COMPLETION DATE

MADE BY
CHECKED BY
APPROVAL BY

CALENDAR DAYS → 5/1/80 … 9/30/80

WORK DAYS → 10 20 30 40 50 60 70 80 90 100

No.	WORK ITEMS	QUANTITY	DAILY PRODUCTIVITY	Schedule (predecessors)
21	South End Bent Form & Pour	30 cyd	10 cyd/day	18, 20
22	South End Bent Cure	-	-	21
23	Order & Deliver Beams	-	-	
24	Set Beams	-	-	10,16, 19,22,23
25	Form & Pour Diaphrams	15 cyd	5 cyd/day	24
26	Cure Diaphrams	-	-	25 25
27	Form Deck And Coping	-	-	26 (3)
28	Rebar	60,000 lbs	20,000 lbs/day	27 (2)
29	Pour Deck W/) Support Cuttouts	150 cyd	150 cyd/day	28
30	Remove Bulkheads & Place Concrete	20 cyd	10 cyd/day	29
31	Cure Deck	-	-	30
32	Form & Pour Top Wall	30 cyd	15 cyd/day	31 (1)
33	Cure Top Wall	-	-	32
34	Reinforced Concrete Approaches	180 cyd	30 cyd/day	32
35	Cure Approaches	-	-	34
36	Place Compacted Aggregate	450 tons	2000 tons/day	35
37	Place Bituminous Mix	250 tons	1300 tons/day	36
38	Bridge Rail	800 lft	600 lft/day	33,35,37
39	Guard Rail	1200 lft	600 lft/day	37
40	Seeding & Sodding	5000 syd	2500 syd/day	37

PROPOSED
ACTUAL

SHRP 80/7 Sheet 3

INDIANA STATE HIGHWAY COMMISSION

CONTRACT Bridge Projects - Sheet 3
PROJECT ______
DISTRICT ______

LETTING DATE ______
START DATE ______
COMPLETION DATE ______

MADE BY ______
CHECKED BY ______
APPROVAL BY ______

CALENDAR DAYS → 5/1/80 ... 9/30/80

WORK DAYS → 10 20 30 40 50 60 70 80 90 100

No.	WORK ITEMS	QUANTITY	DAILY PRODUCTIVITY	Schedule (work days)	
41	Clean-up	-	-	38,39,40	PROPOSED
					ACTUAL

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